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JOURNAL

POWER DIVISION

Proceedings of the American Society of Civil Engineers

# FISH PASSAGE FACILITIES AT McNARY DAM

Glenn H. Von Gunten,[1] A.M. ASCE, Hugh A. Smith, Jr.,[2] J.M. ASCE, and Berton M. Maclean,[3]
(Proc. Paper 895)

## SYNOPSIS

Problems complicating planning and construction of McNary Dam created by fish passage facilities were surpassed only by those of river diversion and control. Although for the most part built integral with the primary project structures, passage facilities for upstream migrants increased the total project cost by approximately $28,000,000, and will continue to represent a major item of project operation and maintenance cost. The McNary facilities are patterned after those provided at Bonneville Dam with modification as deemed appropriate from about 10 years of operating experience and subsequent studies of the problems based on the factual and research data available. This paper provides a description of the facilities, a discussion of the functional design, a resume of construction and operation problems, and a discussion of possibilities for further improvements in design.

## INTRODUCTION

Columbia River is one of the greatest fish producing rivers of the world. It has been estimated that the value at producer levels of the products of the Columbia River fisheries, including that part of the ocean troll to which the Columbia River salmon contributes, is approximately $17,500,000 annually. This industry is supported and maintained by migratory fish populations which use the Columbia River and its tributaries for spawning. Four species of Pacific salmon utilizing Columbia River are: chinook, silver, blueback, and chum. Another anadromous species, steelhead trout, contributes to the commercial catch, but the main value of this species is in the tremendous sports fishing. During the past 18 years the number of salmon and steelhead trout

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passing Bonneville Dam annually has ranged from a low of 361,517 in 1944 to a high of 875,796 during 1952. The maximum peak day passage of 48,582 was experienced 8 September 1942.

The problem of preservation of the commercial and sport fishery resource is common to all major water resource developments on Columbia River and tributaries. McNary is typical of main stem Columbia River and Lower Snake River Projects. Excluding the possible reduction in spawning grounds within the reservoir area, the primary problems are efficient and economical upstream passage of adult migratory fish populations to their spawning grounds and downstream passage of the fingerlings to the ocean.

Two fishways (Figure 1) commonly designated as fishladders, are the basic provisions for passage of the adult upstream migrant salmonoids at McNary. The entrance ways, counting stations, auxiliary water supply and complicated controls are appurtenances. The Washington shore and pressure fishlocks are facilities which are of minor functional significance in providing passage for the large salmon runs. The Washington shore lock serves as an auxiliary facility, since its primary purpose is for experimental research. The pressure fishlock was installed for the single purpose of research, but it can be used under limited conditions as an emergency facility.

In comparison to the facilities provided for upstream migrants, only relatively minor special structural provisions were made for the downstream migrants. However, the problem was a major factor in establishing location, heights of dams, and type of turbines in the over-all planning for development of the Columbia and Lower Snake Rivers. Thus, downstream migrants became a major factor in the basic design at McNary. Special project provisions for interest of downstream migrants include split spillway gates and screening of the Oregon gravity water supply intake.

It is apparent to those familiar with planning, design, construction, and operation of dams on rivers supporting large populations of migratory fish that their efficient passage is a major problem. However, due to the large water resource development program activities by both private and public interests, many engineers are being exposed to the problem for the first time as a major consideration in planning and design. To assist those engineers confronted with problems similar to those at McNary, discussions of the design assumptions are presented in an attempt to distinguish between those criteria based on factual knowledge and those on judgment. This is pointed out to indicate that improvement in design, economy, and efficiency of fish facilities can be attained only by the replacement of judgment by factual knowledge obtained from research and experience.

## Design Criteria

Engineering and construction advancement through experience and research have made large water resource developments possible. Unfortunately, comparative advances in the technique of maintaining anadromous fish runs did not take place. As a result, much factual data for design must be replaced with judgment. In an effort to develop scientifically derived criteria for fishway design, to simplify, and to improve fish passing facilities, the Corps of Engineers has recently initiated an extensive research program specifically directed toward the problems encountered by a construction agency in providing adequate fish facilities at major dams. This program is supplemented by and coordinated with other research programs by Federal and State fishery agencies.

At the onset of planning for McNary Dam, the only successful design and operation of fishways capable of passing major fish runs over a structure of comparable height was at Bonneville Dam. For this reason, the basic design features of Bonneville fish passage facilities were used as a pattern at McNary. Modifications were made to conform with the latest information available and to include equipment and methods that were considered more satisfactory.

The design of all upstream fish passing facilities is based upon the principle that anadromous fish on their spawning migration instinctively swim upstream until they reach the area in which they were hatched. This basic principle is tempered by many factors such as water temperature, turbidity, stream velocity, and chemical content of the water. Since various species of fish may react differently to changed environmental conditions, the reaction of the species concerned must be taken into consideration in a specific fishway design.

In principle, the criteria for the basic elements of the fish passage facilities for upstream migrants may be outlined as follows:

## Criteria for Approach to Fishway Entrances

The major consideration in approach conditions is the development of flow conditions leading to entrances favorable to the natural migration habits and instincts of the fish. A specific definition of ideal conditions is not available. Criteria are generally based on judgment founded on apparent successful conditions at other installations, general observations of migration habits under natural conditions, and known adverse conditions such as slackwater, reverse currents, and extremely high velocities. Probably one of the most effective means of providing satisfactory approach conditions is placement of entrances at locations on the natural migratory paths rather than attempting to create flow conditions to lead the fish from their path to the entrance. At McNary this principle was used in providing main entrances at each shore, an entrance between the spillway and powerhouse, and a collection system with entrances at close intervals across the entire downstream face of the powerhouse. In addition to the strategic location principle, detail design, and location were studied in hydraulic models. Excavated approach channels, special guide wall design adjacent to the spillway, and a large volume of supplemental attraction flow were provided to eliminate adverse hydraulic conditions or to achieve attraction conditions that appeared most desirable. General consideration was given to the total volume of river discharge in the determination of the volume of attraction flow needed.

## Criteria for Fishway Entrances

Although location of the entrances and approach conditions are probably the most significant factors in fishladder design, adequate velocities must be maintained through the entrance to induce fish to continue upstream. Again criteria as to the required depth of flow over the sill and width of entrances are largely based on judgment. Theories of "easy to find" and prevention of "sense of fear or confinement" together with the magnitude of the river, size of the migrant species, and maximum number of migrants expected per day are considered to determining entrance requirements.

## Criteria for Fishways

Fishway criteria are predicated on establishment of conditions for minimum interference with migration instincts in passage from the entrance-way to the forebay. The principal factors are favorable width, depth, velocities, confinement, magnitude of run, and size of species. The height of dam, length of ladder, and the maximum volume of fish to be passed in a day are factors leading to widely varied opinions on width of ladders, length of pools between successive steps, and the desirability of resting pools. At McNary, consideration was given to the density of fish in relation to the volume of water during the period of maximum concentration. Actual density or the maximum daily concentration of fish to be expected at McNary Dam was not known, but estimates were made based on Bonneville counts reduced by estimated spawning in streams between the two dams and estimated commercial catch above Bonneville Dam. Factual data on actual requirements is urgently needed and, except for observed successful passage facilities at existing projects, little more than obvious extremes are known. The physical features of the ladder are designed to eliminate the possibility of injury to the migrants insofar as practical. The relative hazards in fishways to those of the natural stream are unknown and would vary between project sites. Recent experiments and operational observations indicate that ladders providing passage by submerged orifices only may be as efficient as the McNary and Bonneville designs which provide both overflow weirs and submerged orifices for passage from pool to pool of the ladder.

## Criteria for Counting Stations

Provisions have been made for counting of migrants at all fishways constructed by the Corps of Engineers. Until the fall of 1950, a complete count was made of all fish passing Bonneville Dam. Since then, counting has been suspended during winter months when only a very small percentage of the annual fish passage takes place. Although counting provides valuable information, it is an expensive operation, and possibly a deterrent to fish passage. The water passage is 30 feet wide and has a minimum depth of 7 feet at McNary counting stations. Experience has dictated converging the passageway to approximately 4 feet wide and 20 inches deep to provide adequate observation by the counter. Construction to facilitate counting certainly is not the most satisfactory condition for efficient fish passage. Studies have been initiated recently towards developing a less expensive method of counting fish as well as developing counting stations with less confinement. However, due to the many factors encountered, it is doubtful if an economical means will be developed to count without confinement at locations having large runs of several species of fish.

## Criteria for Fishway Exits

The main requirement for a fishway exit is location. It should be positioned in relatively low velocity water and at a considerable distance from either the powerhouse intakes or the spillway. Tagging studies at Bonneville Dam have shown that some salmon are swept downstream over the spillway before they orient themselves for further upstream migration when they are released in the vicinity of the spillway.

### Specific Criteria

Through a series of conferences with representatives of fishery agencies, specific criteria enumerated below were established for design of facilities installed at McNary Dam. The criteria were modified to a minor extent in application to extreme flow conditions of infrequent expectancy.

| | | |
|---|---|---|
| a) | Fishway entrance velocity | 4.0 fps |
| b) | Minimum velocity over submerged weirs | 2.0 fps |
| c) | Velocity through submerged orifices | 8.0 fps |
| d) | Velocity through gross area of diffusion gratings | 0.25-0.30 fps |
| e) | Maximum velocity through gross area of wall diffusion gratings | 0.50 fps |
| f) | Maximum velocity in front of water intake screens | 1.5 fps |
| g) | Minimum discharge from any primary fishway entrance | 1000 cfs |
| h) | Bottom slope of ladder | 1 on 20 |
| i) | Water surface drop at each weir | 12 inches |
| j) | Head on each weir | 12 to 18 inches |
| k) | Width of ladder | 30 feet |
| l) | Height of weirs | 6 feet |
| m) | Number of orifices per fishladder weir | 2 |
| n) | Automatic control equipment sensitive to 0.1 foot change in water levels. | |

## Washington Shore Fishway

The Washington shore fishway (Figure 2) is a reinforced concrete structure approximately 2200 feet long and located between the navigation lock and spillway dam to provide the primary means for upstream migrants along that shore to surmount the dam. Basically, the fishway is a series of pools 20 feet long and 30 feet wide, separated by weirs each one-foot higher than the preceding one. The entrance is at tailrace level and exit at reservoir pool level. The series of pools or total run varies in length from about 1,520 to 1,840 feet, dependent upon tailwater and reservoir levels. Operation within limits of the established design criteria can be achieved for pool elevations ranging from elevation 335 to 342 and between minimum tailwater elevation 248 to a maximum elevation of 274. Maximum tailwater elevation represents a flow of 600,000 cfs and is approximately that which normally occurs during spring floods. A fishlock is also provided at the Washington shore fishway which operates in a similar manner to the navigation lock. The fishlock, a 20 by 30-foot chamber, is not considered an independent facility at McNary since the fish must both enter and exit through the fishladder facilities.

### Entrance Facilities

Adult salmon may enter the fishway over one of four entrance weirs, each of which is 15 feet wide. During normal operation, three of the four entrance weirs are operated simultaneously. Proper design and operation of entrances to the fishway is important since the migrants must be attracted through this passageway. Having passed the entrance, little difficulty is experienced in inducing the salmon to continue up the ladder and into the pool.

Several criteria were established by representatives of both State and

Federal fish agencies for operation of the entrance weirs. These criteria are:

1) The downstream entrance weirs to be automatically operated and sensitive to 0.1 foot in tailwater elevation fluctuations.
2) The minimum depth of flow to be 6 feet over the entrance weirs.
3) The velocity over entrance weirs to be 4 feet per second.
4) The minimum discharge to be approximately 1,000 cfs for any main entrance.

It was determined from hydraulic model testing that the most desirable location for entrance from the spillway stilling basin was approximately 34 feet upstream from the main entrance at a point where velocities would be approximately 10 to 12 fps since fish would have difficulty in penetrating further into the stilling basin. As the tailwater rises with increasing river discharge, the fishway flow must also increase to produce a favorable guiding effect for fish approaching the entrance and to produce adequate velocities in the portion of the ladder submerged by tailwater. To meet this established criteria with a minimum attraction flow of approximately 1,000 cfs, three of the four weirs are operated at a 6-foot depth with a resulting velocity of 4 fps over the weirs. Four telescoping type weirs, each having three leaves 8 feet 8-3/8 inches, operate from elevation 242 to 267, a range of 25 feet. The weirs are raised or lowered by stationary hoists and normally are automatically maintained at a predetermined depth below tailwater by a system of floats, selsyns, relays, and limit switches. Operating experience during the first year's use indicates that the automatic controls are very satisfactory.

Additional features including picketed fish leads, a baffled approach channel and auxiliary manifolds were constructed to provide conditions thought to be optimum at the entrances. Model studies indicated that a baffled approach channel would reduce the approach velocities and thereby improve conditions for fish approaching the entrance at river flows of less than 200,000 cfs. Although excavated in rock, the radial patterned channels all but disappeared during the 2-year construction diversion of the entire river through 12 of the 22 spillway bays. Large flow concentrations with lower tailwater elevations than would normally occur for the completed project resulted in extensive erosion of the rock downstream from the stilling basin and deposition of material in the excavated fishway channel. However, the existing approach conditions, appear satisfactory. Picketed leads, consisting of 1/4 by 12-inch plates set in frames to direct the flow at an angle of 60 degrees from the direction of flow of the spillway were installed in extended piers for spillway bays 1, 2, 21, and 22. (See Figure 2.) Although approximately a 4-inch clear opening exists between the leads, welded-wire fabric on the downstream side of the frame reduces the net clear opening to 1 inch. The purpose of picketed leads is to provide a directional component to the flow to guide the fish toward the side entrance of the fishway. Although operational experience with picketed fish leads at McNary has not been satisfactory, proposed future tests are required to determine the full value that can be derived by their use.

Manifold ports located beneath the bottom of the entrance weirs provide additional flow to assist in obtaining directional control of an eddy caused by spillway discharge and to supplement attraction flow provided by entrance weirs. A 6 by 8-foot gravity supply conduit controlled by a slide gate provides as much as 2,000 cfs to the manifold beneath the three downstream entrances. Gravity flow to the manifold beneath the side entrance into spillway bay 1 is controlled by three 20-inch rotovalves. During normal operation of

the project when river flow is in excess of the power demand, all manifolds are operated to produce the most favorable attraction conditions.

Flow required to fulfill the velocity criterion of 2 fps over fixed weirs submerged by tailwater is supplied by diffusion chambers located in the ladder. Flow is introduced horizontally across a stepped-floor over which bubbler beams and gratings are installed. Model studies and actual operation indicate that the combination of the stepped-floor, the 12 by 12-inch concrete bubbler beams placed transversely to the direction of flow and the grating composed of 1 by 6-inch wooden slats spaced to give 1-inch clear openings and forming the floor of the ladder provide very satisfactory results. The basic criterion limits flow through the diffuser to a velocity of 1/4 fps through the gross area, or 1/2 fps through the net area. This results in a design discharge of 150 cfs for a diffuser located between two fixed weirs 20 feet apart in the 30 foot wide fishladder. The relatively low velocity through the floor area and the close spacings fo the grating were provided to reduce the tendency of salmon from being attracted toward the floor and to prevent entry of fish into the diffuser chamber. A total of 12 diffusers supply gravity flow to the ladder from the area between the entrance and weir 272. Control of flow to these areas is accomplished by 34, 20-inch, rotary-type valves. Because of the disturbing effect of air bubbles on salmon, it was not possible to use air vents on any control valve for the auxiliary water supply system. Based on model tests 19-inch orifices were installed on the discharge end of the 20-inch supply conduits in the expansion wells, to produce sufficient back pressure to preclude cavitation conditions. Although noises emitting from the diffuser piping which resemble a muffled machine gun are excessively loud and challenge the validity of the design, observations of the piping and interior of the expansion wells has not yet revealed any serious cavitation or erosion. A flow varying from approximately 1,000 cfs to 2,500 cfs is supplied through the diffusers depending on the tailwater elevation.

### Fishladder Proper

The fishladder proper consists of a reinforced concrete flume or channel 30 feet wide in which 86 weirs 6 feet high are installed on 20-foot centers. Since the floor slope is one vertical on 20 horizontal, a series of pools having a drop of one foot between pools is produced. Two orifices 23 inches high by 21 inches wide with centerline 3 feet from wall and bottom flush with the ladder floor are located in each weir. Thus, ascending adult salmon may either swim through the orifices or over the weirs. The lowest fixed weir is at elevation 248, the minimum design tailwater and the highest at elevation 333. For identification, the weir number denotes crest elevation of the weir. Tilting and regulating weirs are used above elevation 333 in the exit area. Normal operation at McNary Dam provides a head of slightly in excess of 12 inches on each weir resulting in a flow of 46 cfs through the orifices and 120 cfs over the weir. It was anticipated prior to operation that normal operating head would vary between 12 and 18 inches, but experience has indicated that to increase the head much above 12 inches produces undesirable flow conditions. At higher heads, the flow has a tendency to stream over the pool surface instead of plunging over the weirs. Based on experience at Bonneville Dam and other fishladders, 10 to 12 inches of head on weirs with plunging flow is considered to be the most desirable operating condition. An intermediate flow condition in which some streaming occurs over the center third with plunging

flow at the ends of the weir is also believed to produce satisfactory flow conditions. The fishladder should, also, operate over sufficient range in head to prevent short period pool fluctuations from changing flow characteristics. The most troublesome problem at McNary occurred not with excessive head but with insufficient head on the weirs during the construction period when the water supply was limited by installed pump capacity. When insufficient flow is provided to produce 12 inches of head on the weirs a resonant wave pattern appears in the fishladder. A previous paper on McNary Dam[4] mentioned this observed phenomena. To amplify the discussion of this specific problem it is worthy of note that this undesirable condition was not observed in model studies nor was it thought to be a problem based on operation experience at Bonneville. It is probable that insufficient length of ladder and number of weirs were provided in the 1:16 scale model to establish the surge. The combination of flat-top weir crests, baffling effect of piers within the ladder, changes in ladder width, and the curved alignment of the fishladder effected a reduction in extent and range of occurrence of the surge at Bonneville. Many theories as to cause and measures for prevention of the surge were volunteered when this problem became evident at McNary. However, the type of problem encountered does not lend itself to straight-forward analysis since a combination of both weir and orifice flow are involved. Following limited prototype tests at McNary, model studies were initiated to determine the cause, the most practical remedial measures to eliminate the disturbance at McNary, and the most satisfactory design for fishladders at The Dalles Dam now under construction. Although the model studies are not completed it has been found that no significant reduction in surge could be effected by varying the number of the orifices, the location of orifices, or by removing the bend in the fishladder. Contrary to one early school of thought, elimination of the orifices did not stop the surge, but actually resulted in less desirable flow conditions because the surge remained at a higher head on the weir than for the original design. Several methods of reducing the surge from a maximum of approximately 8 feet in the original design have been found. The surge may be reduced to a minor disturbance by altering the weir crest shape, by constructing alternate pools of different widths, and by installing baffles to reduce the transverse flow across the pools. Separation of the nappe from the downstream face of the weirs was noted for all weir plans which effectively eliminated or reduced the surge. Since nappe separation is influenced by aeration and surface tension which are not reproduced to scale in an atmospheric model using water as the fluid, it is possible that similar action may not be obtained on the prototype. It is quite probable that satisfactory control of the surge for McNary fishladder can be accomplished for lower flows by revision of the weir crest shape. A revision to the weirs at McNary is necessary only if a reduced operating head on the weirs is required as very satisfactory operation is maintained by the automatic flow regulating system located at the upstream end of the fishladder.

In reference to general design of the hydraulic structures, the ladder monoliths are 60 feet in length and provided with copper water stops. Considerable leakage has occurred at many of the joints resulting in appreciable maintenance due to ice during winter months. Leakage has not been observed to date on the Oregon shore ladder where rubber water stops were installed.

4. McNary Dam - Design from Technical Considerations, San Diego ASCE Convention, February - 1955.

The principal exit facilities are the tilting weirs, flow regulating weir, counting station, fishway exit, and a gantry crane for operation and maintenance. The exit area is that portion of the ladder where flow control is accomplished and provision made for counting the fish as they swim into the pool above the dam to continue their migration to upstream spawning areas.

The seven tilting weirs and the regulating weir automatically provide flow control for pool fluctuations between elevation 335 and 342. The tilting weirs provide a necessary continuation of the pool steps and the regulating weir provides the actual quantitative control of the flow for the varying pool elevations. The regulating and seven tilting weirs function as a unit to provide the desired flow conditions. The floor of the fishladder in the exit area is level at elevation 328 from weir 334 through the tilting weir section upstream to the counting station. The tilting weirs are provided with two orifices, 26 inches by 26 inches, and are larger than those in the fixed weirs due to a lesser coefficient of discharge.

The seven tilting weirs, hinged at the bottom, may be rotated through an angle of 90 degrees, horizontal to vertical by two gear racks pivoted at weir top. A single 1-1/2 horse power motor mounted on the deck drives the pinion gears through shaft-coupled gear reducers for each weir. The regulating weir is a telescoping type weir composed of two leaves capable of operating between elevation 333.6 to 341.8 at a nominal speed of 1 foot per minute. The most significant features of this system are that it provides adequate control of the flow and operates automatically for changes in pool elevation. A float-controlled selsyn system provides the automatic control for the entire system including the seven tilting weirs and the regulating weir. As the pool elevation changes, the regulating weir moves to retain its same respective depth below the pool, and the upstream tilting weir also moves to retain the same differential across the tilting weir. When the upstream tilting weir loses control or when a further movement of the weir no longer reduces the water surface, the next tilting weir downstream having a crest elevation one foot lower than the upstream weir will automatically move to provide the required differential water surface on either side of the regulating weir. As previously mentioned, the function of the regulating-tilting weir system is extremely important since rather fine limits are required to provide stable flow conditions in the fishladder. An alarm system is installed to notify the operators of any malfunctioning of the system. This includes power failure, incorrect weir operation and low water. A staff gage mounted on the fishway wall upstream from weir 332 provides a visual check on the operating head on the fixed weirs. Although the system has operated less than one year the results to date have been excellent.

## Counting Station

The counting station is constructed for the specific purpose of counting and identifying species of fish traveling upstream. It consists of a "V-trap" and shelter where a person may observe all upstream migrating fish (Figure 3). The "V-trap" consists of a 4-foot counting board located in the center of the flume flanked on either side by a wing gate assembly forming the "V." Each wing gate assembly is composed of two telescoping gates 5-feet high, a picket panel resting on the telescoping gates and a barrier screen resting on the picket panel. The telescoping wing gates are operated to produce desired attraction velocities through the picket sections and to induce fish swimming

near the bottom to rise to find the opening over the counting board, the only upstream exit from the channel. The wing gates can be operated from elevation 331 when fully telescoped to elevation 341 when fully extended. The picket panels produce the desired directional component toward the walls to guide the ascending fish horizontally toward the counting board. These panels are composed of 7-1/2 by 3/16-inch plates spaced at 1-3/16-inch centers at angle of 30 degrees. Although welded-wire fabric was welded to downstream face of the picket panels, it has been removed because of difficulty with debris. The counting board and exit gate frame are integral and move independently of the telescoping weirs. The louvers of the gate are slanted to direct the flow downward and thereby attract the fish vertically toward the counting board. Closure of the counting station to passage of fish may be effected by either lowering a welded-wire barrier gate or by raising the counting board above the water surface. All control of the counting station equipment may be accomplished from within the counting house, a 4 by 4 by 8-foot inclosure equipped with a 3 by 4-foot window overlooking the counting board. The counting house is hoist-operated from within the inclosure to permit the fish counter to enter the house at deck level and to lower the entire house as close as desired to the water surface. The counting board is painted white and shielded by both vertical and horizontal shades to prevent undesirable reflections and shadows. It is imperative that optimum flow and light conditions be provided at the counting board to prevent delay of fish passage and to assist the counters to identify the fish. It is interesting to note that, in addition to adult salmon and steelhead trout using the ladder, large numbers of other types of fish including lampreys move up through the fishways. During 1954 approximately 310,000 salmon and steelhead trout and 751,000 other fish including 100,000 whitefish were counted as they passed over the McNary counting boards. The counting station is normally operated continuously during the day-light hours, approximately 16 hours, from April 15 to October 15 when the large portion of the fish run normally occurs. Both preceding and following this period for about six weeks, the counting station is operated 8 hours a day. During the remaining months the number of fish passing is insignificant and no count is attempted. Passage through the station is then provided 24 hours a day. The question of whether or not to operate the counting station at night is debateable, but spot checks at McNary Dam indicate only approximately 3 percent of the total count passed the counting board between 8:00 p.m. and 4:00 a.m. The counting station is normally closed during this period by dropping the barrier screen on the board.

Fishway Exit

Exit of fish from the ladder is accomplished through a pair of openings into the upstream navigation lock approach. Trashracks are provided to prevent large floating debris from entering the ladder and yet permit fish to swim through the bars. Although two racks are included, it has been necessary to stoplog one of the openings to attain adequate velocities through the exit. By using one opening the velocity through the gross area was increased from approximately 0.3 fps to 0.6 fps with positive results in moving the salmon from the counting board through the exit. Deflection of floating debris from the ladder exit and trashracks is accomplished by a 5 foot deep concrete beam constructed flush and integral with the navigation lock wall between elevation 341 and 336. Actual operation indicates that this beam should have been

constructed approximately 2 feet deeper to prevent clogging of the trashracks during periods when the pool is below approximately elevation 337. A floating log boom has been installed to overcome this problem.

### Washington Shore Fishlock

During the early planning for McNary Dam it was considered essential that a fishlock be included as one of the fish passing facilities to: (1) provide an auxiliary method of passing fish in case of failure or necessary closure of the north shore fishladder, and (2) provide an experimental facility of improved design which would overcome major deficiencies experienced in the fishlocks at Bonneville Dam. The major deficiency in the operation of the Bonneville Dam fishlocks is the apparent inability to attract fish into the lock chamber. At McNary, this deficiency has been overcome by placing the fishlock in such a position that the fishladder entrance and lower weirs could be utilized as the fishlock entrance. The fishlock is so constructed to permit its operation either with or without the fishladder proper in operation. However, either method of operation requires the functioning of both upper and lower segments of the ladder. The upstream migrants swim from the fishway entrance past the lower weirs, through the lock entrance pool into the fishlock. There they are raised by gravity flow hydraulic lockage similar to that of a single lift navigation lock to forebay level and a channel leading into the ladder immediately downstream from the counting station.

The entrance of divergence from the fishladder is located immediately upstream from weir 252 of the fishladder and is equipped with a 20-foot wide telescopic weir of the same type as the ladder entrance weirs. The crest leaf is surmounted by an arrangement of vertical steel rods curved upstream to prevent fish from returning downstream over the weir. This arrangement is called a finger-trap. Since the weir does not have orifices, the only method of entry into the holding pool is over the weir. A lock entrance pool, approximately 25 by 35 feet in horizontal section, which serves as a holding basin for fish during a portion of the fishlock operation cycle is provided between the entrance and lock. During fishlock operation a continuous water flow is maintained over the entrance weir allowing uninterrupted passage of fish from the fishladder to the holding basin.

The 10-foot wide opening between the holding basin and the fishlock is 30 feet in height to accommodate fish passage throughout wide fluctuation in tailwater levels. The lock entrance is equipped with a ten-foot high entrance gate and a segmented gate consisting of eight 4-foot segments. The elevation of the opening for fish passage may be adjusted in accordance with the tailwater elevation by lowering the required number of segments to the bottom and holding the remaining segments in suspension. This gate also serves as a weir over which water flows providing a passageway for fish into the fishlock chamber. The upstream skin plate is extended approximately 18 inches to provide a sharp-edged weir crest which is considered more desirable for fish movement than a flat-topped crest.

The fishlock chamber, a vertical shaft 20 by 30 feet in horizontal section, provides for an 85 foot hydraulic lift. A diffusion area located in the floor is designed to supply attraction flow through the lock chamber entrance into the entrance pool during "fishing" operations and to distribute inflow uniformly in the lock chamber during filling operations. The diffusion chamber is similar in design to those of the fishladder. The exit gate at the top of the fishlock

chamber, approximately 6-feet wide by 18-feet high, connects the fishlock and exit channel. The gate in the raised position prevents the water in the 10-foot wide exit channel from flowing into the lock chamber. When the lock is filled with water, the gate is lowered so that the top of the gate is flush with the exit channel floor.

An important fishlock feature is the brail, a moveable, slatted lock chamber floor supported by hoist cables. It is used to urge fish from the lock chamber into the exit channel. The slatted floor slopes from three of the outside edges to a small horizontal area immediately in front of the exit gate. Except in the extreme raised position, the brail remains under water. A finger-trap is attached to the brail and positioned just inside the entrance weir. This feature prevents the fish from moving out of the fishlock chamber during periods of fishing.

Water for the fishlock is supplied from forebay by gravity through a 42-inch conduit. This conduit is equipped with a 39 by 39-inch emergency valve in the dam section and a regulating valve of the same size located near the lock chamber. The emptying conduit and valve are the same size as those for the filling conduit. On the fishlock side of the valves, the conduits join to form a common conduit connected to the fishlock diffusion area expansion wall. Conduit sizes were chosen to allow a filling or emptying time of 3 to 4 minutes depending upon tailwater and forebay levels.

A transportation flow of 60 cfs in the exit channel flows into a grillage and sump adjacent to the fishlock through a 20-inch pipe either to the fishlock chamber or entrance pool. If the lock is "fishing," the water enters the lock diffusion chamber and flows over the entrance weir into the entrance pool. When the lock is filling or discharging fish, the water is diverted direct to the diffusion area of the lock entrance pool. This provides a constant water supply flowing from the lock entrance pool to the fishladder.

To provide optimum attraction under certain tailwater conditions, three fishladder weirs immediately downstream from the fishlock divergence from the ladder are equipped with tailwater-controlled telescopic weirs which can be set to maintain a one-foot differential between pools leading to the fishlock. When not in use, these weirs are raised above water level and locked in storage position.

The fishlock can be manually or automatically controlled for operation through the fishing, lockage, and exit cycle on any predetermined schedule. Tests have been initiated to determine the most efficient timing and the relation of timing of operations to the fish carrying capacity of the lock.

### Auxiliary Water Supply

The auxiliary water supply for the Washington shore fish facilities is a feature that provides an interesting hydraulic problem that is not evident from superficial examination. The problem involves the design and operation of a hydraulic system capable of taking gravity flow with a large energy content from forebay and releasing the flow into the fishladder at specific locations at a low energy content. Complicating the problem is the specific requirement that valves must not be vented. In the pressure reducing system provided, water is introduced into three 6-foot by 8-foot conduits through bellmouth entrances having an invert elevation of 271 or 69 feet below normal pool. A 4th intake provides flow through a 42-inch round conduit to the fishlock. Two of the 6 by 8-foot conduits supply flow to the fishladder floor

diffusion areas and the remaining conduit provides flow to the manifold beneath the three main ladder entrances. Trashracks with bars spaced to provide 7/8-inch clear openings are located upstream from the intakes. A hydraulically operated emergency slide gate for each of the four conduits is located in a valve house on the tailrace deck. As the name implies, these valves are not used for regulation and remain in a full open or closed position. The actual control of flow through the conduits is accomplished by 20-inch rotary-type valves. The rotary-type valves are opened and closed from tailrace deck by manual operation of a portable stand. Due to the large gear ratio, the valves are opened by this stand at an undesirably slow rate through the partially-open position. It would be advantageous to provide power-operated stands which would minimize the period of time the valve and piping are subjected to extreme pressure conditions present during the period of opening or closing the valves. Although appreciably higher in cost, the rotary-type valve with smooth, continuous, and unrestricted waterway was chosen to avoid vibration, leaf chatter, and cavitation since design criteria did not permit addition of air into the fishladder. It was considered unsafe to operate conventional gate valves unvented in the part-open positions with velocities in excess of 50 fps. A conventional, 20-inch gate valve was installed in a location adjacent to a rotary-type valve where observation on relative performance can be made. It is not known at present if the gate valve will withstand the extreme flow conditions to which it is subjected. Dissipation of energy is accomplished within a vertical expansion well constructed in the wall of the fishladder where high velocity flow emerges horizontally from two 20-inch pipes to impinge on a steel lined vertical wall. This flow is deflected 90 degrees downward in a section that expands to an area of approximately 50 square feet where it is introduced horizontally into the stepped floor of the diffuser. Satisfactory velocities have been attained in the ladder without excessive boiling or turbulence in the diffuser areas by this system. These valves are not operated in the partially open position.

The method of supplying auxiliary water to the Washington shore fishway is only one of various methods that may be economically utilized and should not be considered the most satisfactory for all applications. Alternate methods that should be considered are: (1) low pressure gravity supply, (2) large low head pumps as provided on the Oregon shore, (3) small low head pumps located at the individual diffusers, (4) turbines capable of utilizing difference in potential between headwater and required pressure gradient in the fishway supply conduit. In selecting the most feasible method of supply, full consideration should be given to: (1) first cost of installation, (2) probable larger maintenance costs of high pressure system over a low pressure system, (3) provision for use of auxiliary water supply system during the construction period, (4) operating costs including cost of power for pumping, (5) value of gravity flow water with full consideration given to the potential loss in power plant capability, (6) and reliability of operation.

## Oregon Shore Fishways

The Oregon shore fishway (Figure 4) is very similar to the Washington shore fishway above weir 253, but the facility below this point is more complicated due to multiple entrances, two auxiliary water supply systems and the actual magnitude of the facility. There are three entrances by which fish are attracted and led to the Oregon shore fishladder: (1) main entrance

(Figure 5) in the non-overflow section of the dam between the spillway and the powerhouse, (2) powerhouse fish collection system, (3) and the Oregon shore entrance. Fish entering the non-overflow entrance are led through a closed fish channel extending across the powerhouse to a junction pool where it joins the powerhouse collection channel and the Oregon shore entrance channel. Fish attracted by the efflux from the draft tubes may enter any of the 44 powerhouse fish collection channel entrances and travel within this channel to the junction pool. Any migrants following the tailrace south shore are attracted by the outflow from the Oregon shore entrance, and after entering it, traverse a short channel leading to the junction pool. Although it is difficult to designate one of the fishladders as the most important, it is generally believed that the Oregon shore facilities will provide passage for greater numbers of adult salmon than the Washington shore fishladder since a large portion of the year all available river flow excluding fishway flow, lockage and other losses, will be used to develop hydro-electric power. Thus, with no flow through the spillway from about mid-August to mid-April, the majority of ascending salmon should be attracted into the Oregon shore facilities increasing the importance of optimum operation of those facilities.

Auxiliary water for all diffusion chambers below 253 is supplied by a pumped, low-pressure supply conduit. Three large pumps, each capable of delivering 2,500 cfs, produce the desired attraction flow. A gravity, high-pressure water supply is provided to the diffusers above weir 253. This combination was chosen in the interest of economy. It also provides additional emergency protection against total loss of auxiliary water supply.

Powerhouse Fish Collection System

This system, the largest collecting unit at McNary Dam, consists of a 17.5-foot wide channel with bottom elevation 236 extending the full powerhouse length immediately above the draft tubes (Figures 6). It is in excess of one-fourth mile in length. Channel entrances from tailrace consist of 88 ten-foot wide openings, 44 of which are blocked with stoplogs and 44 are equipped with automatically operated gates which may be used either as overflow weirs or submerged orifices. Gated and stoplog openings are interchangeable allowing placement of gates at locations most favorable to flow conditions. The channel flow necessary to maintain a 2 fps transportation velocity varies with fluctuations in tailwater. Regulation is achieved by varying the junction pool flow entering the channel at the south end of the powerhouse. Transportation flow varies from 464 cfs at minimum tailwater to approximately 1,300 cfs at a river discharge of 700,000 cfs. Discharge at each gate, either over collecting weirs or through submerged orifices, remains constant at 60 cfs, for a total of 2,640 cfs. Four floor diffusers serve three entrance gates at each main power unit and two diffusers serve the two entrances at the station service units. Flow to these diffusers is controlled by 3 by 3-foot unseating slide gates mounted on the supply conduit outside wall. Model tests indicated that a 2-foot differential could occur in the tailrace water surface sloping from the Oregon shore toward the spillway. To maintain the proper flow from the collection channel, it is essential to maintain water surface slope in the channel parallel to the tailrace slope. Since natural water surface slope for a velocity of 2 fps is usually less than the tailwater slope, the steeper slope required is provided by pairs of vertical-pivoted, wing-gates. Fishery agencies requested that the surface drop not exceed six

inches at each pair of gates; hence, four pairs of gates are provided at equal intervals in the channel to regulate for the maximum expected differential of 2 feet. An additional pair of gates is located at the junction pool to aid in flow distribution. All gates and weirs may be operated manually or automatically float-controlled by water surface fluctuations. At the north end of the channel an automatically-operated telescoping weir provides the required difference in water surface elevation between the collection channel and the main fishway entrance in the non-overflow section. Powerhouse collection system entrance gates at McNary are now being operated as shallow submerged orifices since operational tests at Bonneville Dam seem to indicate such operation is more effective in fish passage than overflow weirs.

### South Non-Overflow Fishway Entrance

This entrance (Figure 5), very similar to the entrance of the Washington shore fishladder, has three 15-foot wide openings leading to a large entrance pool with an alternate 15-foot entrance through the spillway guide wall from the stilling basin. Only three of these entrances are operated at one time. All entrances are equipped with automatically-operated, float-controlled, telescoping weirs. Although the south non-overflow entrance is nearly identical structurally to the Washington shore entrance, these weirs are operated to produce a specified differential in water surface above and below the entrance weirs where as the Washington shore weirs are maintained at a constant depth below tailwater. The normal differential will be 3 inches to produce the required 4 fps velocity.

The diffusion chamber located in the entrance pool immediately upstream from the entrance weirs is the largest in the Oregon shore system with a design capacity of 870 cfs and is supplied through five 4 by 6-foot sluice gates located on the upstream chamber wall releasing water into an expansion well to dissipate excess energy. From the expansion well the water rises and passes along a transversely-stepped floor of the diffusion chamber which deflects the flow upward through bubbler beams and entrances pool floor gratings. The water supply is from the fishway pumphouse.

### Closed Channel

The "closed" or "back" channel, 16-foot wide with a bottom elevation 242, conducts fish from the south non-overflow entrance to the Oregon shore junction pool. This channel paralleling the collection channel, which theoretically could serve the same purpose was considered to offer too many opportunities for fish to return to tailrace through collection system entrances while traversing the length of the powerhouse. Operational tests are being made to determine the effectiveness of the facility. Such tests may indicate the actual need or lack thereof for the "closed" channel at McNary and at similar projects with exceptionally long collection channels. Since the channel joins two pools in which water surfaces differ by more than that required to produce the desired flow, six pairs of wing-gates of the same design and purpose as those in the collection channel are installed.

### Methods of Operation

Transportation flow provided by the junction pool through the closed channel to the non-overflow entrance becomes a part of the entrance attraction

flow. However, two methods of disposing of transportation flow from the powerhouse fish collection channel may be used. One, the so-called "open" system, allows flow to pass through picketed leads to the south non-overflow entrance pool where it may be utilized as entrance flow. Pickets prevent fish from entering the collection channel and guide them toward the closed channel. The "closed" system operation requires closing of the telescoping weir at the north end of the powerhouse channel and spilling of transportation flow over the last two collection channel weirs. In the interest of conserving water, it was envisioned during planning of the structure that normal operation would be the "open" system unless disturbances caused by flow through the pickets detracted from the non-overflow entrance efficiency.

Oregon Shore Fishway Entrance

This entrance provides direct access to the junction pool and fishladder for fish following the south shore of the tailrace. It has two 15-foot openings equipped with telescoping weirs designed and operated the same as those in the south non-overflow entrance. Immediately upstream from these weirs, a side-wall diffusion chamber is provided. This was necessary because there was insufficient space for a floor diffuser. Since the water surface at the junction pool must be at least 12 inches higher than at the entrance to maintain the required flow through collection system weirs or orifices, two pairs of wing-gates are provided to control flow from the junction pool to the Oregon entrance.

Junction Pool Area

The junction pool area located at the south end of the powerhouse is the confluence of the Oregon shore entrance channel, the powerhouse collection channel and the closed channel. This junction of channels is one of the most important areas of the passage facilities since all migrants using the Oregon shore ladder must pass the area. From the standpoint of hydraulics, the correct flow distribution in the entire system downstream can be maintained only if correct water surface elevations and flow conditions are maintained at the junction pool. Above the junction pool, the ladder proper begins. The first four pools of the ladder are divided into two channels to provide additional diffusion area for the large quantity of water required at the junction pool and to provide an entrance channel for a fish lock which may be desired in the future.

Automatic Control Equipment

A central control system is provided for the 44 collection system entrance gates, wing-gates, and main entrance weirs. The equipment is designed to automatically actuate and control the weirs and gates to maintain the desired relation between water levels in the fishways, junction pool and tailwater. Although the relative elevations of the water surface in the system can be automatically maintained, automatic control of flow to the diffusers is not possible. The automatic control is accomplished by a-c control bridge circuits utilizing transmitting slide-wires which give gate and water level references to control bridges. The bridges are in electrical balance as long as the gate and water level conditions correspond to the conditions set into the bridge control circuits by calibrated, dial-operated rheostats. Changes in water level from the

desired differential unbalance the bridge circuits, and resulting voltage differentials are detected by electronic relays. These detection relays then operate the gate or weir motor contactors which in turn operate the gate machinery until the bridges are again in balance. The installation is expected to be completed and placed in operation this fall (1955). Operation at present is accomplished by manual adjustment as often as necessary to maintain the weirs at the desired elevation with respect to tailwater. The top of the 1-1/2 foot high by 8-foot wide orifice is presently set 2 feet below tailwater by moving the weir until the calibrated dial at each weir indicates the desired elevation.

### Fishladder Proper

The primary features of the Oregon shore fishladder above weir 253 are identical with the Washington shore facilities previously described. A fish viewing area consisting of four 2-3/4 by 4-3/4 foot, 1-1/4 inch polished plate glass windows were inserted in the fishladder wall between weirs 305 and 307 and enclosed in a building. The major portion of the light entering the building is from the windows in the ladder to aid visibility but the entrained air and turbidity of the water prevents observation of many fish. However, during certain periods of the year multitudes of lampreys sticking on the windows provide an interesting sight and occasionally salmon may be seen.

### Exit Area

The exit releases ascending migrants into the pool approximately 1,300 feet from the powerhouse, a sufficient distance to eliminate the hazard of salmon entering powerhouse intake passages. Exit area equipment including seven tilting weirs, a regulating weir and counting station is almost identical to that previously described for the Washington shore. Intakes for both the gravity diffuser water supply and the future fishlock water supply conduit are constructed integral with the structure. The intakes are screened by large traveling screens later described in conjunction with the auxiliary water supply system. Control of flow is accomplished by 22, 20-inch, rotary-type valves located at the expansion chambers. An 8 by 8-foot tainter gate installed just downstream from the traveling screens provides emergency control and is normally operated in either full open or closed position. Since present plans do not indicate that the fishlock will be constructed in the near future, the State of Washington is tentatively planning on utilizing this conduit to provide flow for an extensive spawning bed test ground in the area downstream from the project.

### Auxiliary Water Supply System

The larger portion of the supplemental water added through diffusers below weir 253, is pumped from tailrace. A pumphouse with three pumps, each capable of providing 2,500 cfs at 6 feet of head, is located on the south shore immediately downstream from the powerhouse. The pumps discharge into a pressure conduit 22 by 24 feet leading toward, and branching four ways under the junction pool. A branch follows under each side of the fishladder upstream from the junction pool, a third branch to the Oregon shore entrance and the fourth to the powerhouse diffusers and diffuser No. 1 at the south non-overflow section. Flow to the respective diffusers is controlled by 93, 3 by 3-foot,

unseating, sluice gates and 5, 4 by 6-foot, sluice gates at diffuser No. 1. The 3 by 3-foot sluice gates are hydraulically operated by a portable pump. Sluice gates at diffuser No. 1, however, are limit-torque motor-operated. Discharge through the system is controlled by varying the number of pumps operating, changing the pump inpeller angle, and by changing diffuser gate openings. Although the system is very complex and a variation in any gate settings effects, to some degree, the flow to remainder of the system, flow conditions seem rather insensitive to minor variations. The major operational difficulty lies in the fact that to affect any desired variation requires lengthy re-calculation of gate settings. Since measuring devices were not provided the hydraulic analysis of the entire system must be based on the manufacturers model efficiencies, power consumption, and theoretical hydraulic computations. Since the maximum efficiency of the pumps is considerably below the rated discharge, a major portion of the time all three pumps are operated to take advantage of higher efficiencies. Pumping requirements vary from 4,200 cfs for minimum tailwater conditions based on a river flow of 43,000 cfs to 5,500 cfs for a river flow of 600,000 cfs. The average discharge is about 4,600 cfs based on a river flow of 180,000 cfs.

Gravity supplemental flow is provided for an average period of five months each year when tailwater is in excess of elevation 256. The seven diffusion chambers for the Oregon shore fishladder that are supplied by an 8-1/2-foot round conduit are of the same type previously described for the Washington shore installation. These diffusers, however, have three 20-inch rotary-type valves instead of two as provided on the Washington shore although the diffuser floor areas are equal. An 18-inch orifice is installed on each 20-inch I.D. pipe to avoid pressure indicated by model tests to be in the cavitation range. At the lower end of the gravity conduit an interconnection between the high pressure gravity conduit and the low pressure pumped supply conduit is controlled by a 72-inch Dow Disk valve. This valve is normally closed, but it may be opened in event that the pumped water should fail. Computations indicate that approximately 1,000 cfs may be supplied through the interconnection to the low pressure diffusers.

To prevent fingerling salmon from being drawn into the conduit and injured in the expansion wells of the diffusion chambers, traveling intake screens are provided. The two, 10-foot wide screens, the largest commercial size available, extend from elevation 275 to the operating deck elevation 348. It is estimated that for the maximum flow of approximately 1,300 cfs, the mean velocity will be 1.4 fps through the net area. The screens are operated at time intervals as required to keep them free of debris. Operation can be either automatic or manual. An alarm system is installed to indicate excessive head on the screens. Since fingerlings attracted to the intake by the flow of water but are screened from passing through it, a means of passage over the dam is required. This is provided by a collecting well 2.5-feet square in each of the two outer piers of the intake structure into which the fingerlings may pass through a series of 6-inch diameter ports. These ports are located on 30-inch centers immediately adjacent to the screens. The fingerlings may then pass into the fishladder by a 30-inch pipe discharging into an open well on the downstream side of the regulating weir. From this point, the fingerlings swim down the ladder. Operational tests to determine apparent effectiveness of the fingerling by-pass facilities are now in progress.

## Pressure Fishlock

Efficient passage of migrants directly through a dam from tailrace to forebay without moving them to pool level would be advantageous at dams having a wide fluctuation in forebay level. An experimental pressure fishlock was installed in the south non-overflow section to determine the efficiency of this method of passing the fish. The decision to change the method of auxiliary water supply for the powerhouse collection system from, gravity to pumped source at McNary Dam provided a limited area in the non-overflow dam between the spillway and the powerhouse suitable for the experimental fishlock installation. The installation consists of a short approach fishladder leading from the fish entrance at the north end of the powerhouse, a fishlock chamber, and an exit channel to the forebay. The approach fishladder is eight feet wide with a bottom slope of one foot in fifteen with pools fifteen feet long. The weirs between pools consist of two hand operated telescoping leaves engaged in slots in the fishladder walls.

The 4 by 11-foot fishlock entrance gate is operated in the inverted position to function as a telescoping overflow weir when the lock is fishing. A sharp crested weir plate was placed on top of the gate to improve hydraulic conditions. A fingertrap was also included to help retain fish in the fishlock when in a fishing position. Thus, the fish enter the lock chamber in a similar manner to that in the Washington shore gravity fishlock. The lock chamber, twelve feet long and 8 feet wide, varies in height. The slanted roof of the chamber joins a 4 by 6-foot tunnel about 22 feet long leading to forebay. This tunnel is closed at mid-length by a motor-operated standard sluice gate designed for balanced-head operation. The exit tunnel has two openings into forebay beyond the gate, one through a vertical 7 by 8-foot shaft to pool elevation and the other a continuation of the exit tunnel direct to forebay. A manually-controlled screen directs the fish to the desired exit. By closing the entrance gate and introducing water from forebay with an air escape vent open, pressure inside the lock equalizes with that of forebay. Thus, the exit gate may be opened with balanced-head conditions. By regulation of the drain valve, a flow of water can be maintained through the fishlock chamber. In theory, fish directed by the water flow move out of the fishlock chamber to forebay in an effort to escape distress from the change in pressure and natural instinct to migrate against the current. Whether this theory is correct has not been definitely determined, but limited operation of this facility has established that the design is functional. After the fish have made their exit the upper gate is closed, the lock chamber drained to tailwater level and the cycle repeated.

The lock chamber is provided with three separately controlled lights, two on the south wall about midway in elevation and the third in the roof. These were provided to aid in observing the action of fish in the lock chamber through an observation window located in the south lock wall.

Water is supplied to the fishlock and approach fishladder through a 4.5 by 4.5-foot intake located in the bottom portion of the 7 by 8-foot fish exit shaft. The conduit size reduces through a constant radius transition to a 24-inch diameter pipe equipped with a hand-operated slide gate. The 24-inch pipe is connected to the diffusion areas of the fishlock and the fishladder by four 8-inch pipes each equipped with a motor-operated, rotary-type valves.

Water to attract fish into the fishlock is introduced through one of the 8-inch pipes to the diffusion area in the bottom of the fishlock. When the

fishlock is filling or discharging fish, this water supply is shifted to another 8-inch pipe opening into a diffusion area in the floor of the upper end of the approach fishladder resulting in a constant flow down the ladder. Since the approach fishladder is twice the width of the lock entrance, additional water is required to maintain the desired flow in the fishladder. This additional water is supplied by another of the 8-inch pipes also leading to the diffuser area in the upper end of the fishladder.

The lock chamber is filled by the fourth 8-inch pipe leading to the diffusion area in the bottom of the fishlock. The air in the chamber escapes through a 12-inch gate-controlled vent. In order that pressure will not build up too rapidly inside the lock chamber, the 8-inch filling pipe is throttled by the insertion of a 4-inch orifice. To aid in future operational studies, the pipe is so constructed that the orifice is removable or can be replaced by another size. The lock chamber is drained by means of a 12-inch drain pipe connecting the lock diffusion area with tailwater.

## Temporary Fishpassing Facilities

Construction at McNary Dam required two major cofferdams. The Washington shore cofferdam, extending approximately one-third of the distance across the river, was the first element of construction. This construction did not increase the river velocities sufficiently to materially interfere with fish migration past the project. Therefore, no means were provided to aid in fish passage during this stage of construction.

The second phase of construction included a cofferdam extending from the Oregon shore approximately two-thirds of the distance across the river connecting with the Washington shore cofferdam. Prior to closure of the main or second phase cofferdam, the Washington shore cofferdam was removed. The bays of the spillway dam built within the Washington shore cofferdam were left unfinished at a low level to provide a passageway for the river flow after closure of the main cofferdam. These bays were capable of passing river flows up to about 370,000 cfs without causing opertopping of the main cofferdam.

Plans for the second phase construction period included three temporary fishladders for fish passage. Incorporated into the wooden cofferdam cribs and forming an integral part of the main cofferdam through spillway bay 13, a temporary wooden fishladder 20 feet wide was provided. This vertical-slot fishladder contained two 29-inch wide slots in each baffle. The principle of vertical slot fishladder design limits its use to locations where tailwater and headwater variations are similar. To compensate for uneven variations in headwater and tailwater existing at McNary, a gated orifice was constructed in each baffle. A more uniform drop through the baffles was attained throughout the fishladder by adjustment of these gates. The slope of spillway bay 13 temporary fishladder floor was one foot vertical on 19 feet horizontal with the ladder design providing a minimum of 6 feet of water in all ladder sections under expected low flow conditions. The ladder entrance was positioned to take advantage of the eddy created by the main cofferdam and the high velocity water flowing through the spillway dam. The upstream end of bay 13 fishway turned shoreward along the upstream leg of the main second-step cofferdam for a short distance to provide fish exit under favorable conditions away from the high velocites and drawdown into the spillway area.

A second temporary fishladder was located on the face of the north training

wall of spillway bay 1. This fishway was 12 feet wide with a slope of 1 foot vertical on 21 feet horizontal. Twelve vertical slot baffles were utilized with one slot to each baffle to result in a ladder approximately one-half the size of bay 13 fishladder. The ladder entrance, determined by hydraulic model studies, was located at the upstream point of the return eddy and downstream from the trough of the hydraulic pump.

Since the cofferdam on the Oregon shore was expected to flood each year, an additional ladder facility was provided at the shore end of the upstream leg of the cofferdam to allow fish to escape upstream during cofferdam inundations. The cofferdam construction was such that during overtopping fish could enter the cofferdam area over the downstream leg but could not escape over the upstream leg without the fishladder.

The temporary fishladder constructed in spillway bay 1 failed due to the excessive violence of the water passing through the spillway bay. To facilitate passage an emergency trap was built in the partially completed fishlock. The fish were lifted over the dam by a crane-operated bucket. This facility was used throughout the remaining construction period until the permanent Washington fishladder began operating. It is estimated that as many as 5,000 fish per day were passed over the dam in this manner. Although there may have been minor delay and injury to some of the migrants, the operation was successful. It is not apparent that the runs have been affected to any appreciable extent.

To permit the shortest practicable interruption of navigation and still allow completion of upstream construction of relocations and levees, an interim pool level was maintained during the summer of 1953. During this period water was pumped into the upper end of the permanent Oregon fishladder. This enabled the fish to use permanent facilities in ascending to the upper end of the fishladder where they were collected and lowered to forebay in a pair of buckets. The water supply was furnished by two 27,500 gpm pumps and directed into the ladder by a wooden diffusion grating built into the floor of the permanent fishladder. Each of the two buckets was constructed of rolled steel plate, was cylindrical in shape, and had a capacity of about 1,600 gallons. A double-compartment, reinforced-concrete chamber was provided integral with and just outside the permanent fishway structure. The chamber served as a collection area or trap with the fish buckets forming the bottom portion of each chamber. These collection chambers were equipped with diffuser gratings which serve the double purpose of distributing the water flow and channeling the fish into the buckets. The system was arranged for alternate bucket operation with one bucket in the raised position collecting fish during the time the other was transporting fish down to the forebay pool. A portion of the pumped water was alternated from one collection chamber to the other by a manually-operated, remote-control, valving system. This operation was very successful with no apparent delays or adverse affects on the migrants.

The importance of temporary fish passage facilities during construction should not be overlooked from the initial planning stage throughout construction. Fish passage during construction may be a controlling factor in the overall planning of the project, design features of cofferdams, permanent facilities, and construction scheduling. Failure to provide adequate facilities during construction could cause irrepairable damage or even total elimination of a major fish run. Consequently, designers should not yield to the temptation of skimping on the design due to the anticipated short period of use. Consideration must also be given to the fact that it may be very difficult to

anticipate all construction problems, conditions, and delays that may be encountered which would affect the adequacy of temporary fish ladder design. Adequate design for the fixed or controlled operating conditions of the permanent facilities is generally not so difficult to provide.

## SUMMARY

Based on the operational experience to date, the fish facilities at McNary Dam appear fully adequate for efficient passage of upstream migrants. Operational tests are in progress to evaluate the efficiency of various alternate methods of operation provided for in the design. Recently initiated research programs should add materially to the factual data available for the establishment of specific design criteria for efficient passage and economical design of future projects. Tests were conducted at McNary to determine the mortality of downstream migrant fingerlings during passage through the spillway and the turbines. Over a million marked fingerlings were used in these tests during the past year. Neither the test operations nor the analysis of the tests are complete to date. It may be found that this years tests are of more significance in evaluating testing procedures than of an accurate evaluation of mortality. Should accurate evaluations of mortality be possible and the specific cause determined, the results could be of major significance in the design of future water resource development projects on streams supporting major populations of migratory fish.

FIG. 1 MCNARY DAM

FIG. 2 WASHINGTON SHORE FISHWAY

FIG. 3 COUNTING STATION

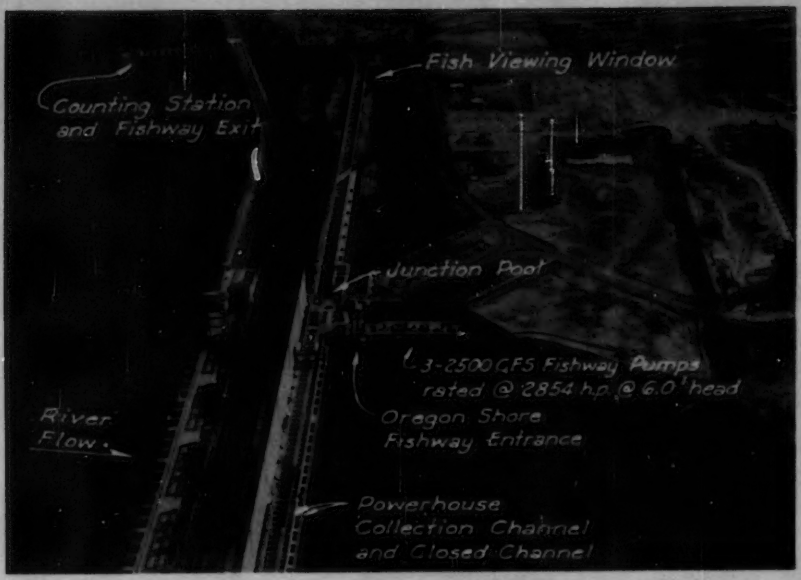

Fish Viewing Window
Counting Station and Fishway Exit
Junction Pool
3-2500 CFS Fishway Pumps rated @ 2854 h.p. @ 6.0' head
Oregon Shore Fishway Entrance
River Flow
Powerhouse Collection Channel and Closed Channel

FIG. 4 OREGON SHORE FISHWAY

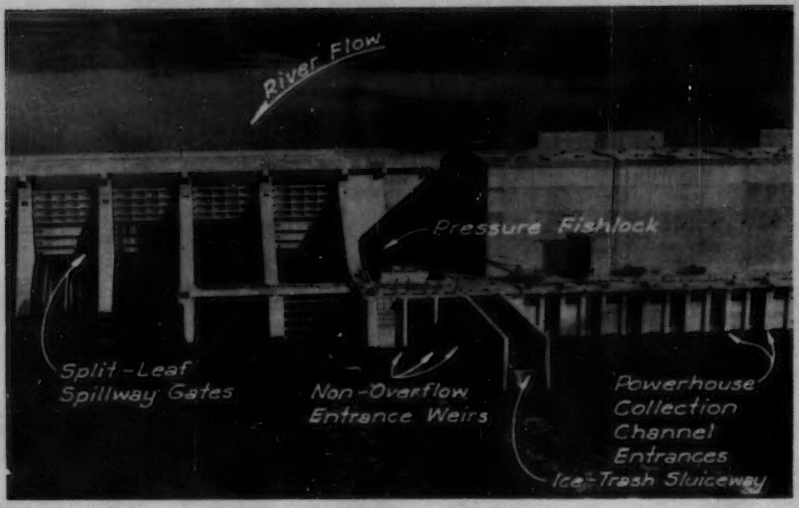



FIG. 5 NON-OVERFLOW ENTRANCE

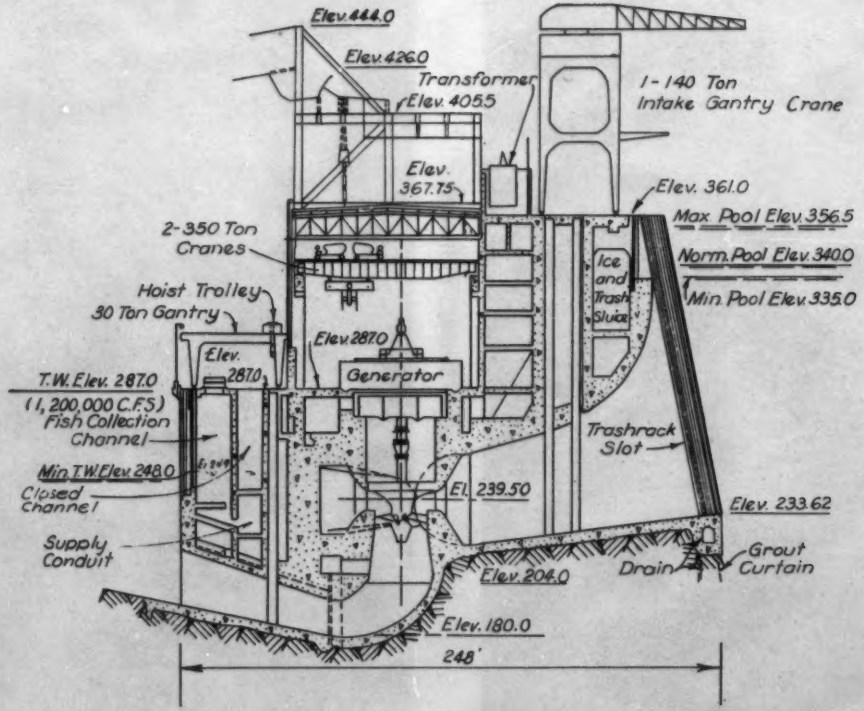



FIG. 6 TRANSVERSE SECTION THROUGH POWERHOUSE

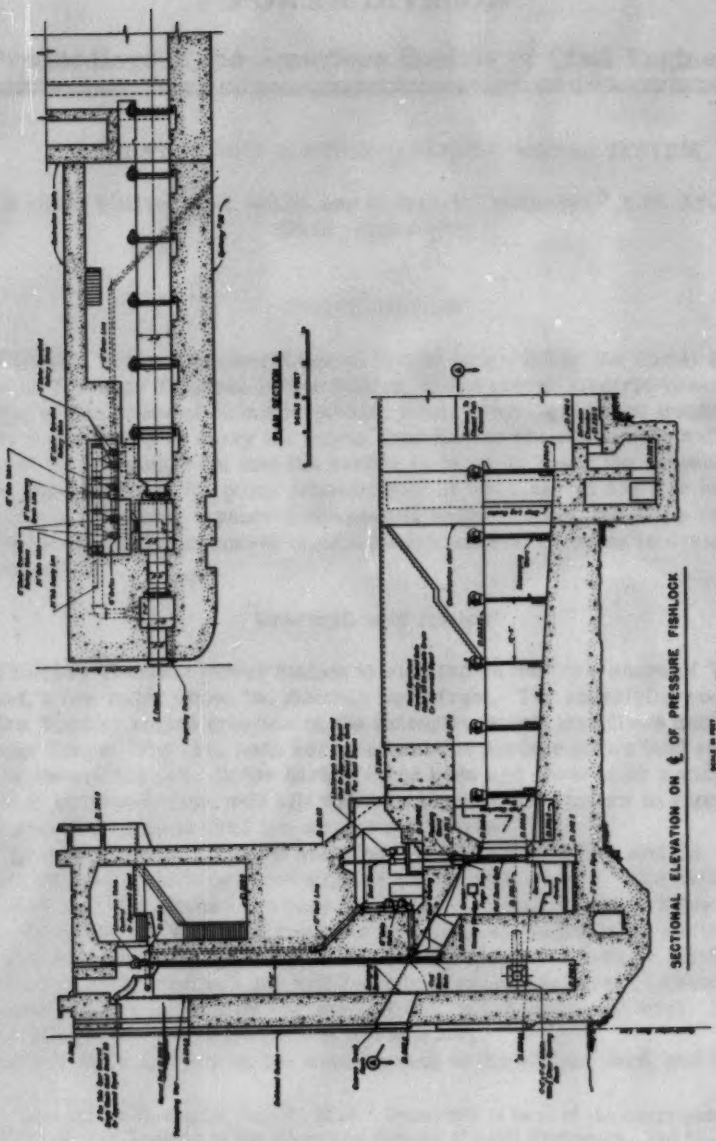
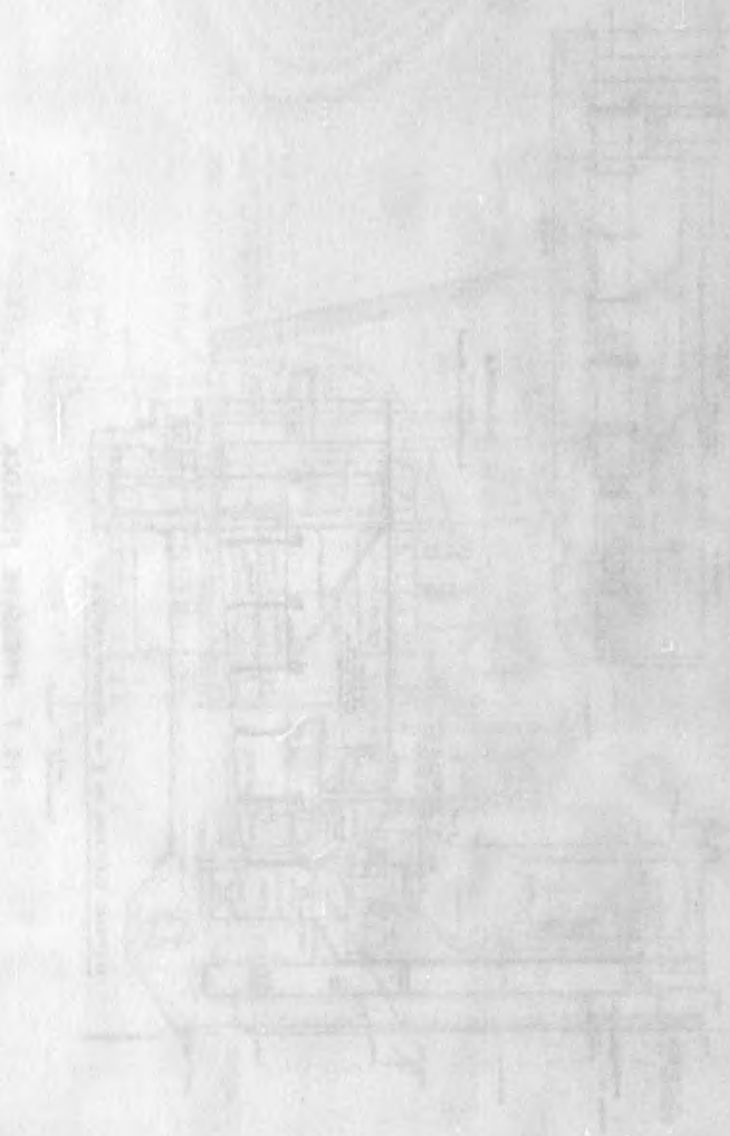



FIG. 7 PRESSURE FISHLOCK



JOURNAL

POWER DIVISION

Proceedings of the American Society of Civil Engineers

# TROMBAY POWER STATION: COOLING WATER SYSTEM

William Wallace,[1] M. ASCE, and George E. Archibald,[2] A.M. ASCE
(Proc. Paper 896)

## INTRODUCTION

The Tata Power Company Limited is now constructing the initial development of Trombay Thermal Power Station, a new steam-electric generating station on the shore of Bombay Harbor, India. Physiographical conditions make it necessary to carry the intake conduits for the condensing salt water system nearly a mile out into the harbor in order to reach the required minimum depth of water for pump submergence at low tide. In addition to constituting a relatively expensive condensing water system, the pump intake and conduits and conduit supports involved some unusual features in design and construction.

### Description of Station

Trombay Thermal Power Station is situated on the fore-shore of Trombay Island, a few miles above the Bombay waterfront. The underlying rock over all the Bombay region consists of the extensive basalt lava flows known as the Deccan Traps. The lava beds here lie near the surface of the land and dip gently toward the sea. In the harbor these beds are covered by a varying depth of extremely fine, soft silt which is too soft and shallow to support conventional driven piles used for marine structures.

The power station is on the northwest shore of the harbor and the orientation of the line of turbine-generators is nearly north-south. The eastern side of the station lies against the base of the Trombay hills. Most of the station area, however, is reclaimed from the tidal flats by means of an earth fill which raises the yard three feet above the highest recorded tide level, this level being about nine and one half feet above mean sea level. Lowest recorded water level is about nine and one half feet below mean sea level. The mean range of spring tides is eleven and one half feet.

Across the tidal flats on the western side of the station yard, and forming

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1. Chief Engr., Sir Bruce White, Wolfe Barry & Partners, London, England.
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a south-western boundary to the yard, is an embankment, or bund, the older portion of which has for many years supported oil lines from a ship unloading pier belonging to Bombay Port Trust. The pier, of solid masonry construction, is located on the edge of an area of relatively deep water, called Pir Pau Deep, where ocean-going tankers are able to unload. This older bund terminates at low water line and is connected with the pier by a light structural steel trestle. These unloading facilities will be in large part supplanted by an extensive new oil terminal on Butcher Island, now under construction, with submarine pipe lines connecting with Trombay Island. The new lines will continue ashore on a widening of the old bund on the west side. Adjacent to this bund on the Trombay Power Station side, a drainage channel lined with hand placed riprap has been constructed to carry storm and plant drainage to the harbor from Trombay Station and the Standard Vacuum refinery area to the north. This channel is designed to carry the estimated maximum of 3,000 cfs storm drainage, at high tide.

Annual rainfall at Trombay averages about 80 inches, all of which falls during the monsoon period; in general from June to September. The intensity of the storms hampers construction work and it is often necessary to entirely suspend operations during this period.

The initial steam-electric plant development at Trombay comprises two 62,500 kw turbine-generator-units. Each unit has its own boiler and auxiliaries and has a cross condenser. The building basement extends down to 13.5 feet below yard level, while the operating deck floor for the units is 21 feet above yard level. The building is completely enclosed up to the operating deck but above this only the machines are enclosed. Boilers are outdoor type. Initial operation will be with oil fuel but provision is made for future conversion to coal firing, with the coal handling and storage facilities to be added at time of conversion. Space has been reserved for these purposes when they are required. Road and railway access have been provided.

The two initially installed turbines will operate at 1250 pounds per square inch and 950 degrees Fahrenheit at throttle. Power will be generated at 13,800 volts, 50 cycles which will be stepped up to 110 kv in three phase transformers, and will leave the switchyard on distribution feeder lines at this voltage.

## General Design of Condensing Water System

Condensing water requirement for the two initial units at Trombay is 167 cubic feet per second. This will be carried in one 72 inch diameter conduit at a velocity of 6 feet per second. In the design of the pipe supporting structure provision has been made for accommodation of an additional conduit of the same capacity for future generating units.

Soundings in the silt floor of the harbor indicated that to reach the required minimum depth of 15 feet of water at low tide it would be necessary to go to about 2000 feet beyond low tide shoreline. This is approximately one mile from the power plant.

The combination of rock at shallow depth with its overburden of soft, shifting silt ruled out the possibility of a pump station inshore served by intake canal, or gravity conduit. The alternative was a pump intake structure at deep water with pressure conduit to the plant. These conditions and the economics involved also made it impracticable to lay submarine lines of the size and number required out to the intake structure. The design indicated

was a pipe-supporting mole or bund of substantial construction out to the economic limit due to depth of water, and a supporting pier, or jetty from this point to the intake; these structures to be of sufficient height to raise the pipes above wave action.

## Joint Facilities

Other simultaneous industrial developments in the area had to be considered in the design of the water intake system. Burmah-Shell and Standard Oil Companies were each constructing a new refinery in the vicinity of Trombay Power Station, each requiring a salt water intake and intake pipe lines to shore. Bombay Port Trust were already well along in their plans for the "take-off" in this area of the submarine oil pipe lines to the new oil terminal to be constructed on Butcher Island. Discussion between the representatives of the various parties interested suggested considerable savings as well as avoiding of interference, by construction of joint facilities where possible. As a result these facilities now comprise approximately 1300 feet of joint rockfill bund extending into the harbor to support the new Port Trust oil lines and the circulating water lines from Trombay Power Station and the Burmah-Shell refinery to a point where the oil lines diverge to become submarine; and a farther length of approximately 1600 feet of bund and reinforced concrete jetty which support the Trombay and Burmah-Shell water lines to their respective intakes at low tide. These joint facilities are designed to carry a future additional 72 inch pipe from the Trombay Station.

## Model Study

As a prerequisite to obtaining permission from the Bombay Port Trust to erect structures out into the harbor, it was necessary to show evidence that these structures would not cause undesirable changes in the configuration of the harbor floor due to their effect on tidal flow. A model study was made in the model laboratory of Sir Bruce White, Wolfe Barry & Partners, London, the firm of consulting engineers who were retained to design and supervise the construction of the joint facilities. The model reproduced the portion of Bombay harbor around the proposed location of the new bund and jetty. Model scales were 1 to 400 horizontal, 1 to 150 vertical, 1 to 12.24 velocity and 1 to 32.6 time scale for tidal phenomena. A wave maker simulated waves up to an amplitude of five feet. The structures simulated comprised a bund extending 2000 feet, out beyond low water shoreline, and a reinforced concrete jetty, 1000 feet farther and terminating at an enlarged jetty head with two adjacent intake caissons. The structures were placed in optimum locations as indicated by actual expolorations in the harbor which were still in progress.

In the series of tests, model runs were made representing 10 days of spring tides. Study of the results of all the tests combined with study of the history of Bombay Harbor indicated that the structures proposed would have no appreciable effect on the natural movement of ground material in this area of the harbor. Actually, history indicates that silt has been accumulating here in recent years, a condition which made it advisable to allow some margin over the minimum depth of water required at the intakes, in order to postpone the time when dredging will be required.

Final Arrangement

Bund, jetty and intakes have been constructed very close to the alignment studied in the model. The final arrangement of the circulating water conduits and supports for Trombay Thermal Power Station, starting from the station building and running in a southerly direction is as shown on Figure 1 and is briefly as follows:

A length of 777 feet of double barreled cast-in-place reinforced concrete tunnel with two 72 inch diameter circular barrels in vertical configuration, the top barrel being the discharge and the bottom the intake conduit. This conduit is buried, resting on rock at the level of the turbine room foundation mat. At the southern end of this run of conduit the top or discharge tunnel diverges to the west at an angle of 57 degrees and continues 312 feet to a seal well and discharge structure at the storm drainage canal. The intake continues south for 105 feet and rises to the ground surface to form an anchor for the north end of the concrete lined 72 inch steel pipe which then continues above high water to the branch connections to the pumps at the intake for a pipe length of roughly 4100 feet.

For 800 feet south from its junction with the concrete tunnel the steel pipe crosses tidal flats within the Trombay station property and is supported on concrete saddle piers resting in an earth bund. The relatively shallow fill is superimposed on the soft native clays and silts of the flats so that short end-bearing concrete piles are used to carry the pier loads down to firm ground. The southern end of the run of pipe is fixed by an anchor block which also extends down to rock. This 800 feet of pipe was not made a conventional run of continuously welded construction with a packed expansion joint in the run because of the lateral forces which would be transmitted to the piers and piles by sliding of the pipe due to changes in temperature. Accordingly, the pipe is fabricated in 42-foot sections which are field connected by Dresser couplings. In this way the expansion forces will be in equilibrium between anchors with small movements at each coupled joint. The concrete support at each field joint consists of two saddle piers on a common footing, one pier on each side of the coupled joint. One pipe end is anchored to its pier, the other end is free to slide. An early stage of construction is shown in Figure 2, with the new earth bund in the center of the picture. The joint rock fill bund is in the right background and Pir Pau bund and jetty run from right to left near the top of the picture.

From the southern anchor of this pile-supported run the pipe angles slightly to the west and crosses the storm drainage canal and the older Port Trust oil pipe bund. The crossing is on a long diagonal and requires 408 feet of pipe between anchors. Six bridge type concrete piers at 60 foot intervals support the pipe at intermediate points. These piers are carried down to rock and are designed to accommodate a future additional line. The pipe is continuously welded here and is designed to be self-supporting between piers. It also carries a service walkway and the electric cables running to the intake. Rocker arms attached to ring girders on the pipe reduce lateral thrusts on the free standing piers due to expansion and contraction of the pipe and a packed pipe expansion joint is provided at the southern anchor.

The southern anchor of the drainage channel crossing marks the junction with the jointly-constructed rock fill bund, described later. The anchor is carried down to rock as are the other channel crossing supports. As the pipe proceeds south along the rock fill, however, anchors and piers are seated in

the fill and are subject to whatever construction-residual and storm-induced settlement may occur after the pipe is installed. To provide flexibility for differential settlements, field lengths of 42 feet of pipe connected by Dresser couplings were again used. On the concrete jetty, since the jetty piles extend into rock and the structure is therefore rigid, the pipe is continuously welded and a packed expansion joint is provided at each expansion bay in the jetty, generally at intervals of 400 feet. A pipe anchorage is provided at midpoint of each interval to resist unbalance in pier friction and in the forces required to operate the expansion joint at each end. A saddle pier supports the pipe on each pile bent and a graphite lubricated compressed asbestos sheet is provided on the pier to reduce sliding friction. At the junction of bund and jetty a hinged section of pipe will permit the shoreward end to settle without bringing excessive loads on the adjacent jetty bent.

## Conduit Design

In the design of the intake conduits and supports a maximum internal hydraulic pressure of 60 pounds per square inch pump shut-off was used. This produces a large end thrust for two 72 inch pipes which must be resisted by the jetty head bracing as described later. Water hammer pressures, which otherwise could be much greater than this pressure will be controlled by vacuum breakers. The 72 inch diameter steel pipe has 7/16 inch plate reinforced with girth rings to resist distortion and vacuum loads. The pipe is lined with spun cement lining one inch thick and the exterior is painted with red lead paint.

At the intake four 36 inch diameter branch pipes will receive the discharge from four vertical pumps, each in a separate chamber and each protected by a traveling screen. The water will be chlorinated in front of the coarse screen as it enters the intake structure in order to prevent marine growths on screen and the conduit system. Power for the pumps and other equipment will be carried out to the intake at 3300 volts in cables laid in concrete trench along the bund and jetty.

## Design and Construction of Bund

The joint bund, or embankment, which extends into the harbor to about 1100 feet beyond low water line, is of dumped rock construction, and is shown during construction in Figure 3. Its total length is approximately 2200 feet. The minimum top width of 70 feet is occupied by the Trombay Steam Electric Station and Barmah Shell refinery water lines, the Port Trust oil lines, and a 12 foot roadway.

The bund cross-section is zoned, the rock sizes running from Grade D with 4 to 12 inch sizes to Grade A with blocks 18 to 36 inches in least dimension, each zone having prescribed proportions. The Grade A rock constitutes the protective facing against wave action on side slopes which are 1 on 1-1/2. The nose of the bund, from which the reinforced concrete jetty springs, is revetted by a mastic asphalt grout poured into the joints of the facing blocks.

The top of the fill is five feet above maximum recorded high tide in Bombay Harbor. Material was quarried from the trap rock in the adjacent hills and was transported to the fill by dump truck. The initial fill was extended by end dumping and was carried three feet below finished grade, leaving this depth for later special consolidation. Hand placing of rock was limited to the

pitching of the face, although some stone was carefully placed by crane at the nose of the bund to avoid damage to the concrete piles of the jetty. In the area where the Port Trust oil pipes diverge to become submarine a length of roughly 800 feet of bund is gradually widened to accommodate the "mud pond" for the oil pipes, the Trombay and Burmah Shell water lines continuing at the higher level. The bund terminates at the depth where economy dictated the change to reinforced concrete trestle, this depth of fill being roughly 35 feet.

The depth of silt overlying the rock bottom of the harbor varies, being 6 to 8 feet-generally along the bund. The "A" Grade dumped rock displaced this silt satisfactorily, so that the rock fill rests on firm bottom. Where the silt was deeper than 6 ft. to 8 ft. it was dredged out before placing stone. Maximum observed settlement of the fill after 5 months was 3-1/2 inches, occurring at the nose of the bund.

## Design and Construction of Jetty and Jetty Head

Exploration was made of the harbor floor by means of borings and soundings preceding the general routing of bund and jetty. Additional closely spaced soundings were made to find the most advantageous location for the jetty head and intake structures. The rock floor has a characteristic stepped formation, due to the eroded laminations of successive lava flows, which have a westerly dip in the order of 5 degrees. At the surface the rock varies considerably in hardness; also this surface has been at some time incised with a complex system of stream channels, later filled with conglomerates or disintegrated basalt. Consequently there are appreciable changes in depth to sound rock within short distances, viewed from the standpoint of structure foundations. The value of close borings and probings in such formations cannot be too strongly emphasized.

The jetty and jetty head, jointly occupied by Tata and Burmah Shell, is of reinforced concrete construction. Sections are shown on Figure 4. The jetty generally consists of 37 four-pile concrete trestle bents on 20 foot centers, the bents being connected by a concrete beam-and-slab deck containing a roadway 10 ft. wide with cable trenches on each side of the roadway. The bents are composed of four precast piles, two raking, to brace the frame laterally, connected at deck level by an in-situcapping beam. The piles are 16 inches by 21 inches in cross-section and the feet are square ended and fixed in sockets cut in the rock by the Benoto "hammer-grab" process, described later. The top width of this trestle structure as measured by length of pile cap is 47 feet. Running roughly in a southerly direction, the two 72 inch steel water lines to Trombay Station will occupy the trestle on the east of the roadway and the Burmah-Shell Refinery water lines occupy the west side.

The trestle is designed to carry the operating pipe loads mentioned, plus a roadway loading equivalent to a 10 ton truck in each bay. The pipes are supported at the pile caps only. The trestle is designed to be stable against 10 foot waves, assuming one half ton per square foot against the projected area of the piles for this height of wave. The elevation of the deck is 8 feet above maximum recorded high water, so that no forces other than wind are assumed laterally against deck and pipes.

Three expansion bays are provided in the 770 feet of trestle, one of which is at its junction with the bund. The central bays of each resulting section of trestle are braced longitudinally by means of raking piles. Because of the expansion joints placed in the pipes where they cross the trestle expansion bays

and the anchorages at the central braced bays, the bracing is designed to resist any differentials in the thrusts required to operate the pipe expansion joints on either side.

At the seaward end the trestle expands into a tee-shaped head roughly 210 feet long, and 81 feet wide in the common central roadway portion. The intake structures fit into the easterly and westerly seaward corners of the jetty head, permitting future expansion of intake facilities for each party in opposite directions without interference. The jetty head deck is of reinforced concrete beam and slab construction and is supported on the same type of piles as the jetty. Deck level at the jetty head is about 48 feet above the rock floor of the harbor.

The Trombay Station portion of the jetty head is designed to carry a live load of 300 pounds per square foot, which covers pipe and equipment loads and incidental live loads. An important item in the design of the jetty head bracing was the forces imposed by the terminal anchorages of the 72" diameter Trombay pipes. The maximum simultaneous hydraulic thrust from the two pipes plus the thrust required to operate two expansion joints, was estimated at 488,000 pounds. Sufficient brace piles were provided to take this thrust and the deck slab was thickened to 18 inches in order to provide weight to resist the upward components of these pile reactions against the deck.

The Benoto "hammer-grab" tool used to excavate holes in the rock for seating the piles is essentially a heavy hammer with a head in the form of a strong "orange-peel" type of digging bucket, with three jaws which can be locked open to form a chopping bit for hard material, then unlocked to permit them to be operated for removal of the broken material. So far as is known, the first use of this equipment from floating support was in construction of the jetty and jetty head and similar work being done simultaneously for the new Port Trust oil terminal at Butcher Island. Figures 5 and 6 show construction operations, with "hammer-grab" in a raised position in Figure 7.

Piles for the jetty and jetty head were cast on shore and transported to the site by barge. At the site, the typical pile setting operation required the seating of one vertical pile and one brace pile in the same bore hole in the rock. The construction procedure was as follows:-

A 48 inch diameter open ended steel casing, the "Benoto tube," was sunk through the silt to the rock floor at the site of the proposed borehole. The Benoto tube was held in position by a light latticework box girder, supported on and run out from the piles already placed. Operating within the Benoto tube and through the water the hammer grab was used to excavate the silt and then to break and excavate the rock to the required depth. The two piles were then lowered through the tube into the excavation. The tube was then removed, the piles being supported temporarily by the box girder guide. After removing the tube, the raking pile was pulled over into its correct position and both piles braced in place with a system of rolled steel joists clamped to and run out from the piles already placed. Finally the Benoto hole was filled with a colloidal, that is, intimately mixed, grout, by the "Colgrout" process.

A 4 inch diameter hole was provided through the middle of each pile for the introduction of this grout. However, as the grouting had in any case to be inspected by diver and the grouting plant was carried on a pontoon, which at low water might be 25 ft. below the top of the pile, it was found quicker and more satisfactory in practice for the diver to grout up the excavation through a pipe introduced beside the piles. The diver was thus able to stop the grouting when the hole had been exactly filled and avoid waste of grout or the possibility of

grouting being stopped before the hole was filled. A period of 12 hours was insisted upon to allow the grout to set before excavation of the next set of holes was permitted, as considerable vibration was transmitted through the guide frame to the newly grouted piles during the boring process.

Occasionally, groundrock irregularities met during excavation would cause the Benoto tube to shift off the correct position or to go down on a slight rake, a tendency that needs watching. Once the hole had been cut it was found that, with the size of pile in use, very little adjustment of the position of the piles could be made. In future operations of this type, it would be an advantage to allow more clearance for the piles within the Benoto hole.

An adequate depth to which the piles should be seated was assumed at 3 feet in sound rock. The rock surface, however, in some places is decomposed or of soft structure. At such locations the boreholes were carried to greater depth and where the depth of hole interfered with pulling the raking pile into position the bottom of the hole was filled with a concrete plug up to the desired level for stepping the piles, sometimes by placing concrete in three quarters filled Hessian bags, the tops being sewn, instead of tied, to avoid possibility of settlement. Since the piles were precast in a limited number of lengths, the tops were trimmed to the proper elevation after setting, as a matter of routine.

The use of the Benoto hammer-grab proved very satisfactory. It took a little time to familiarize the operators with the special equipment, but this was not a serious delay. In sound basalt, progress in excavating holes reached an average of about one foot per hour or 12 holes per week. Special measures required in soft rock or for other reasons of course prevented this average from being realized at all times. Additional supporting equipment for the pile setting operations included floating derricks and concreting equipment.

Deck portions of jetty and jetty head were cast in place. Wood formwork, or shuttering, was supported by strutting from the rolled steel joists which were employed to brace the piles in position before grouting, as shown in Figure 7.

The concrete strength specified was 3750 psi at 28 days in field tests as determined by crushing 6 inch cube specimens. The sand came from Karsalia and crushed rock aggregate was a microgranite which occurs on a limited scale at Chandivali.

The aggregates were selected after considerable research, the microgranite being specified for all marine work in preference to the regional basalts because of the better shape produced in crushing and because of its freedom from "Chlorophacite" and allied, chemically unstable, constituents. The Karsalia sand was somewhat deficient in fines, but this was offset by an excess of fine particles in the coarse aggregate which had been obtained by jaw-crushing. All site concrete was consolidated by internal vibration. The wetting agent "Lissapol" was used in the mix to increase workability. The average cube strength obtained was in the order of 6500 psi at 28 days.

Two concreting ships were used for placing concrete, one equipped with a small steam crane and the other with a fixed gantry carrying a hand operated overhead conveyor. The concreting ships were kept supplied by a fleet of barges.

Design and Construction of Intake Structure

The Trombay Station intake which is designed only for the initial installation of two generating units, is a simple box structure of reinforced concrete, 42 feet by 48 feet in plan with four compartments, one for each pump. The structure is seated on a leveling bed of rubble about 4 feet thick which, in turn, rests on rock bottom. Dumped rock aprons around the structure protect against erosion. The floors of the pump chambers are 15 feet below low-low water and 41 feet below deck level, the deck being flush with the jetty head deck, which extends along the north and west sides of the intake and is separated by an open joint.

The intake structure was designed to be constructed in dry dock as a caisson, which would be floated to the site and sunk to position. The top 8 feet of structure was omitted for the caisson stage of construction; also for flotation purposes, the design wall and floor thicknesses were reduced, with provisions for increasing these thicknesses after sinking in order to provide weight for stability. The intake openings in the front walls of the pump chambers were closed with temporary steel plates and temporary valves were provided for use in flooding the chambers for the sinking operation. Figure 8 shows the caisson nearing completion in drydock.

Preparation of the site consisted of dredging away the overlying silt, cleaning the rock surface by air-lift and depositing and leveling the rubble bed. These operations were checked by a diver. The top of the rubble was dressed off with the regard to the differentials in settlement expected due to the varying depth of rubble.

During favorable weather conditions on February 13th, 1955, the caisson was towed to the site. Control of position during sinking was by five pontoon-mounted winches on east, west and south, and two steam winches on a floating crane to the north. The caisson was closely positioned at flood tide and the final positioning and sinking operation was accomplished during slack water within three hours. A check showed a deviation of two inches to the north and one inch to the east. Positioning of the caisson is shown in Figure 9.

The weight of the incomplete intake structure without the top 8 feet is roughly 2140 tons as compared with 988 tons water displacement with all pump chambers open at maximum recorded high water, and 2013 tons water displacement with all chambers dry and the same water level. Pending completion of the upper 8 feet the valves are being left open. The structure is designed to withstand wind and 10 foot waves and also will act as anchorage for the pipe header with its branches to the pumps, this header being free to slide on the deck of the jetty to avoid transmitting forces to the jetty head other than those transmitted at the main terminal anchor previously described.

## GENERAL

At this writing, the bund, jetty and jetty head have been completed and work has started on construction of the top 8 feet of the intake structure. Construction of the underground cast-in-place concrete conduits at the station building has been completed, also construction of anchors and pier supports for the steel pipe along the bund. Trial operation of Trombay Thermal Power Station is scheduled for the summer of 1956.

## ACKNOWLEDGMENTS

Thanks are tendered to the owners of the various structures described for permission to publish this paper, and to all the engineers associated in the work for their cooperation.

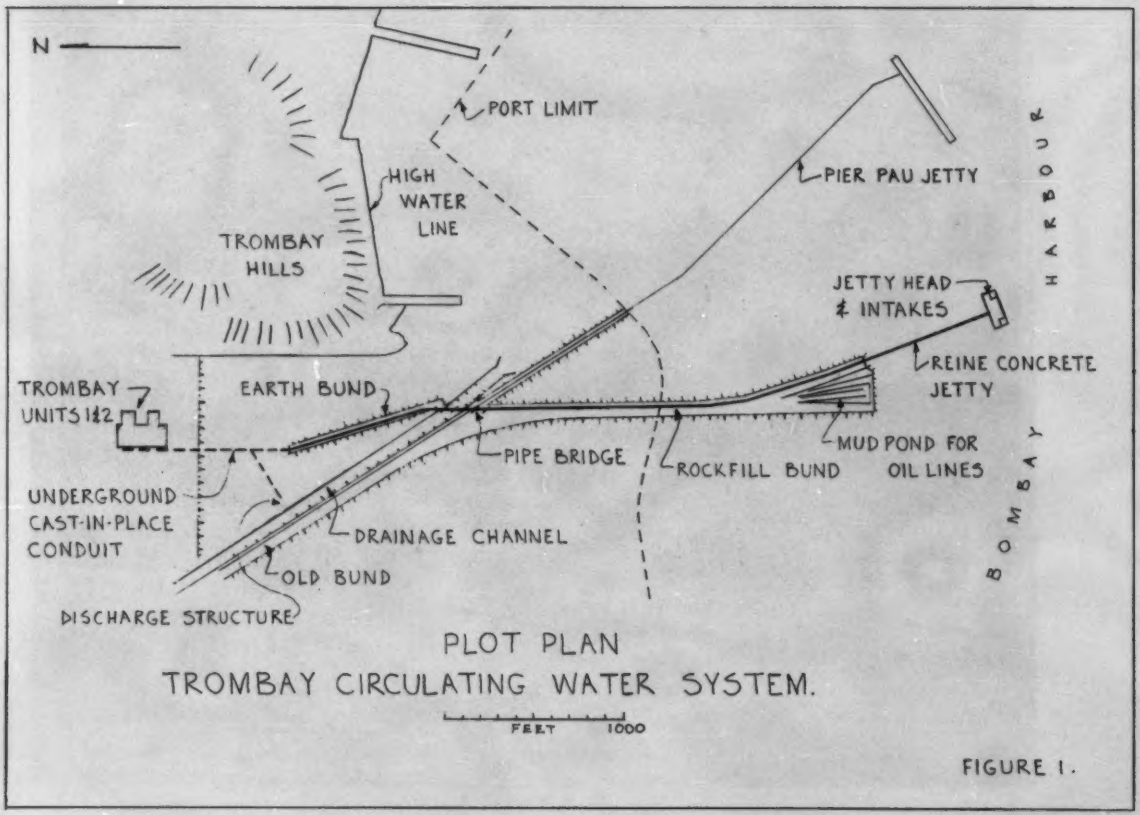

N
PORT LIMIT
HIGH WATER LINE
TROMBAY HILLS
PIER PAU JETTY
HARBOUR
JETTY HEAD & INTAKES
REINE CONCRETE JETTY
TROMBAY UNITS 1&2
EARTH BUND
PIPE BRIDGE
ROCKFILL BUND
MUD POND FOR OIL LINES
BOMBAY
UNDERGROUND CAST-IN-PLACE CONDUIT
DRAINAGE CHANNEL
OLD BUND
DISCHARGE STRUCTURE
PLOT PLAN
TROMBAY CIRCULATING WATER SYSTEM.
FEET 1000

FIGURE 1.

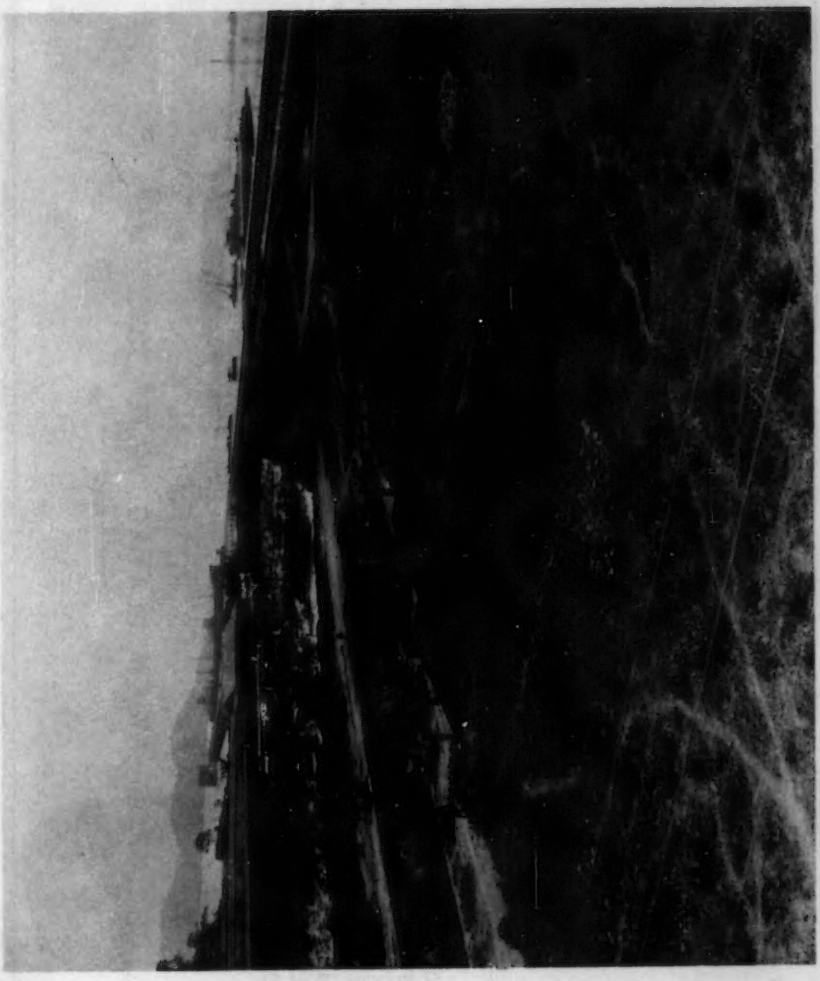


FIGURE 2

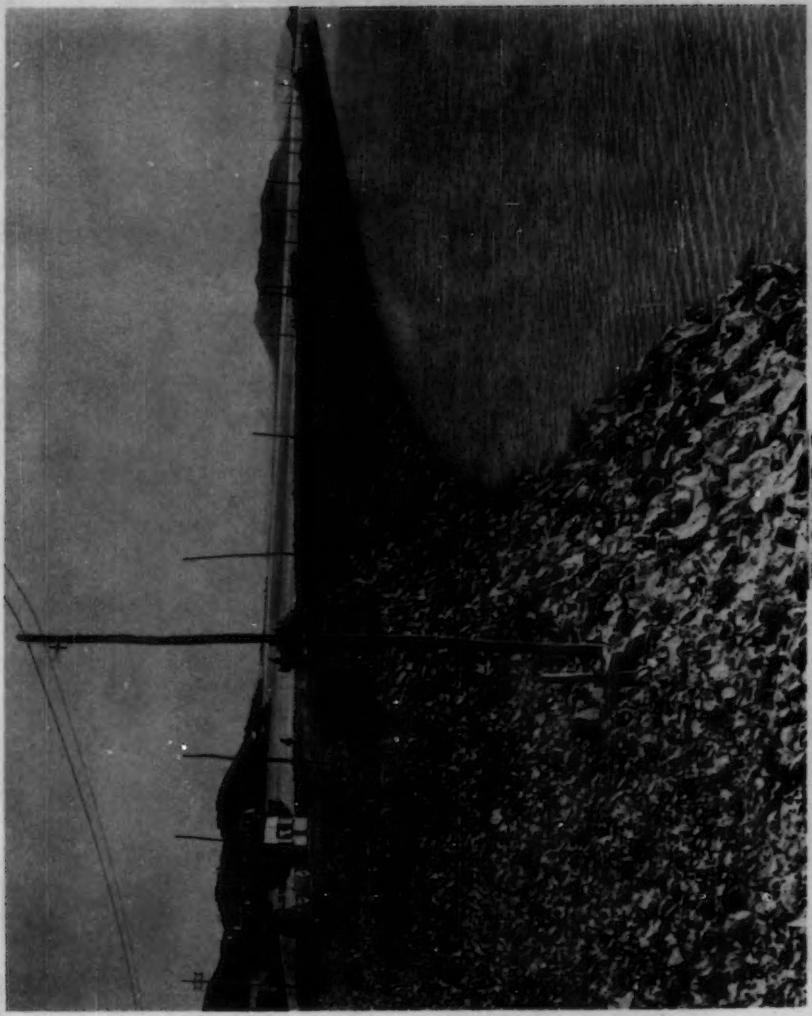


FIGURE 3

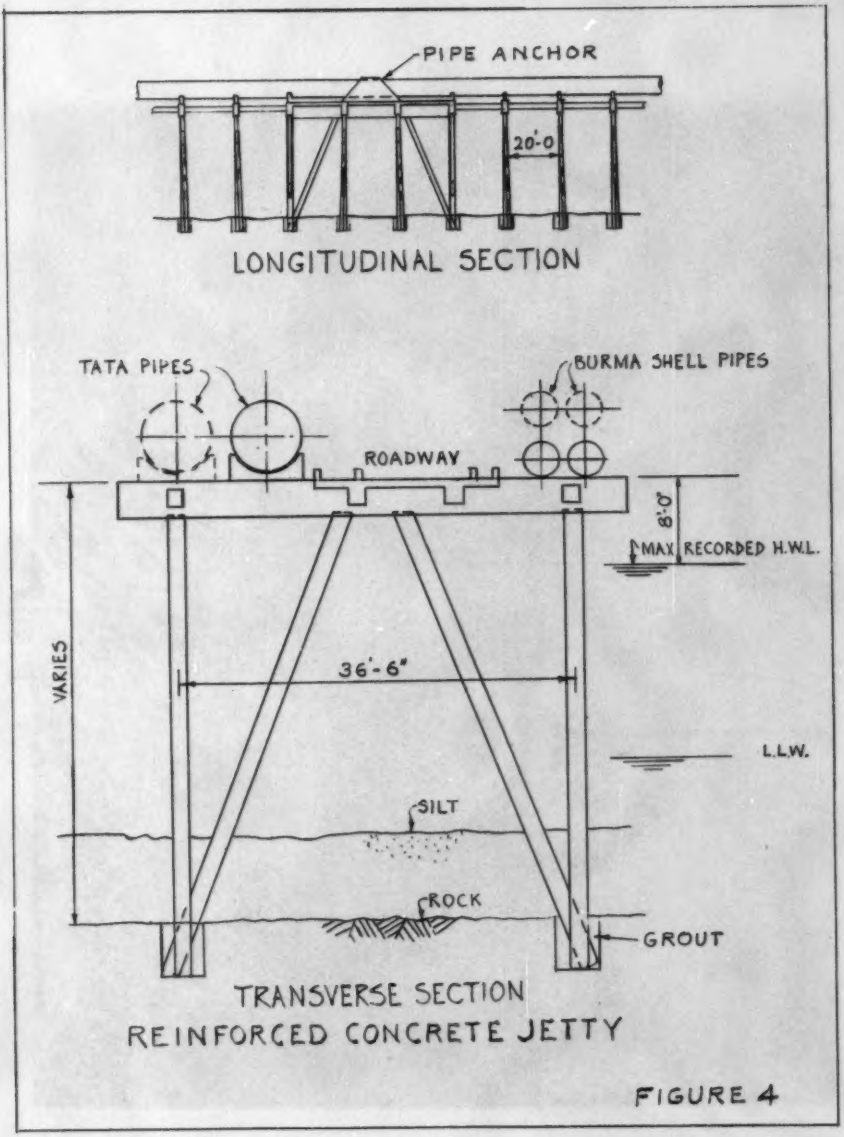

PIPE ANCHOR
20'-0
LONGITUDINAL SECTION
TATA PIPES
BURMA SHELL PIPES
ROADWAY
8'-0"
MAX. RECORDED H.W.L.
VARIES
36'-6"
L.L.W.
SILT
ROCK
GROUT
TRANSVERSE SECTION
REINFORCED CONCRETE JETTY

FIGURE 4

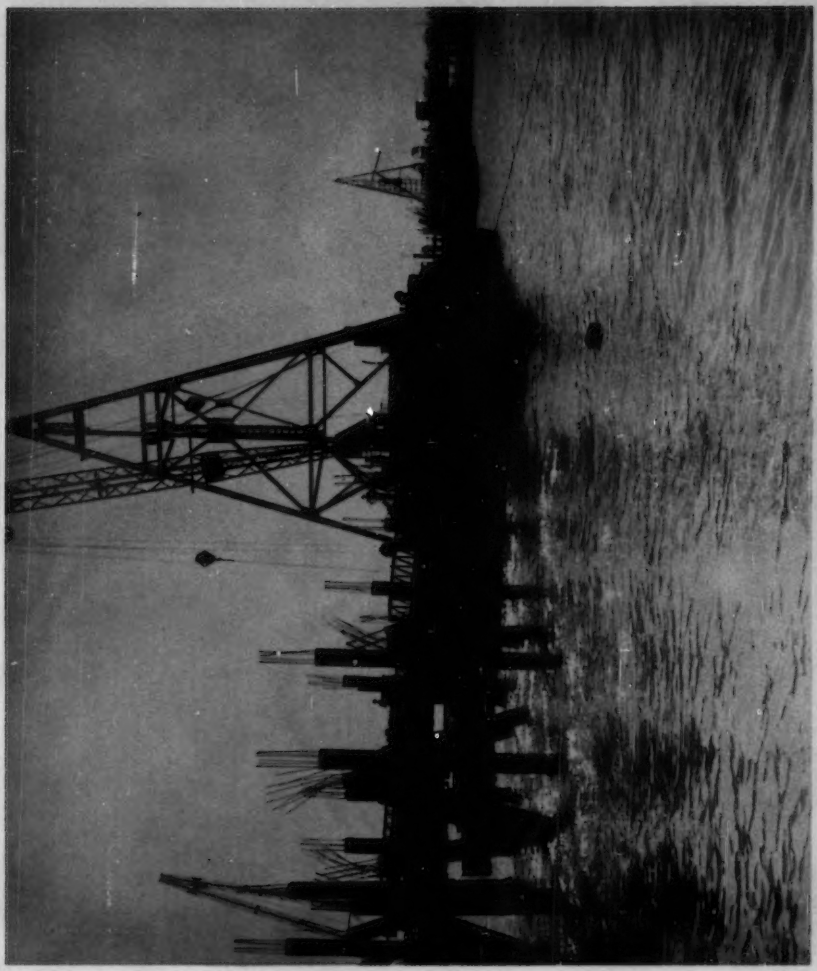
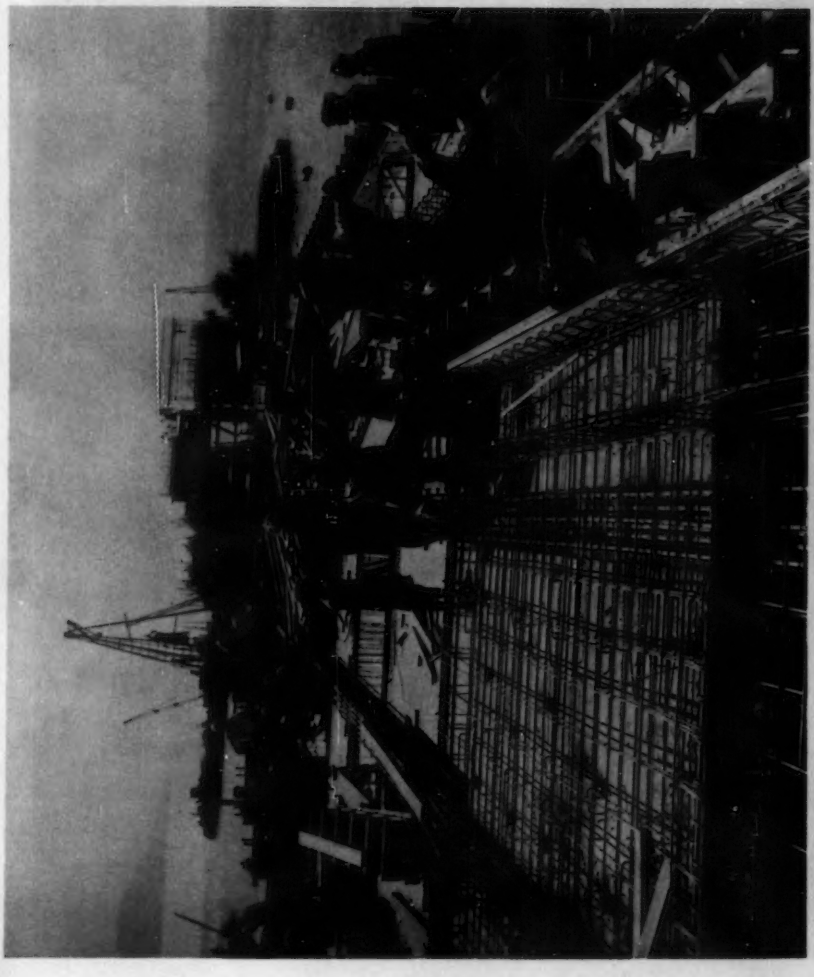


FIGURE 5



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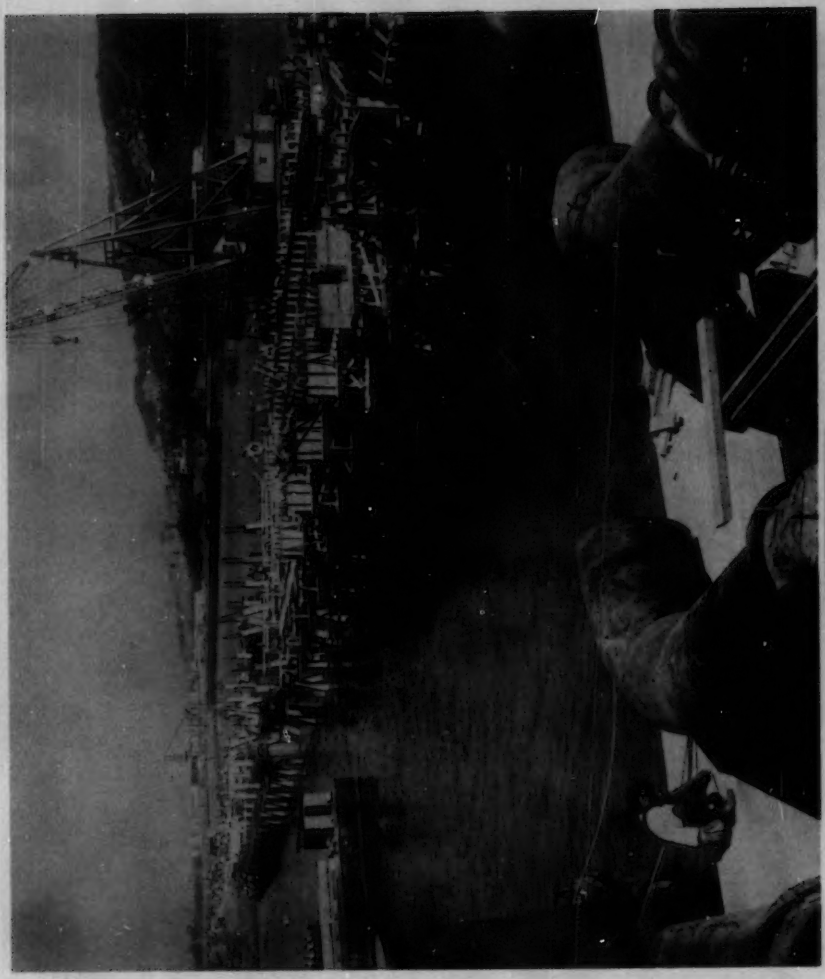


FIGURE 7

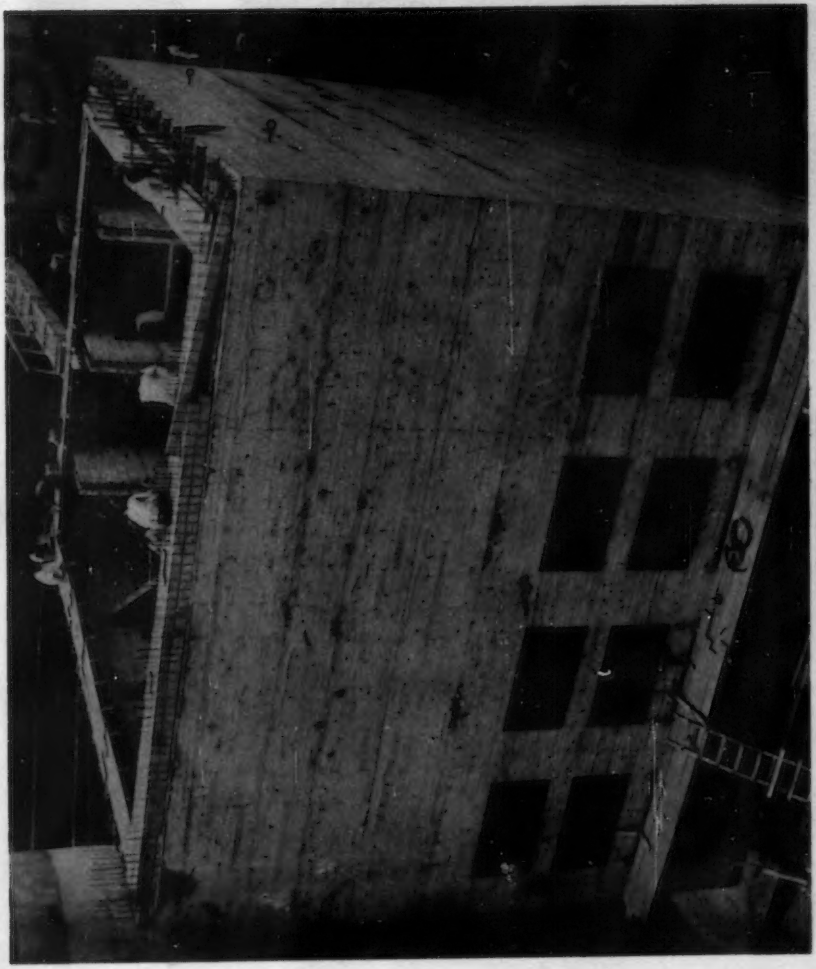


FIGURE 8

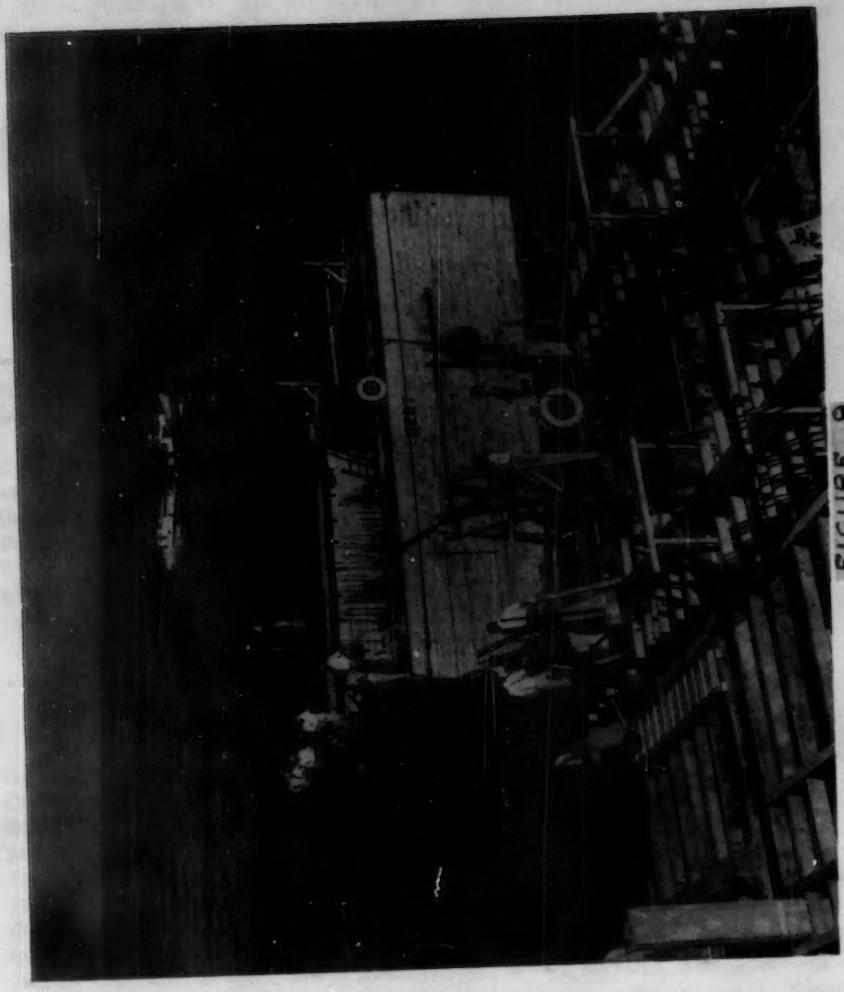
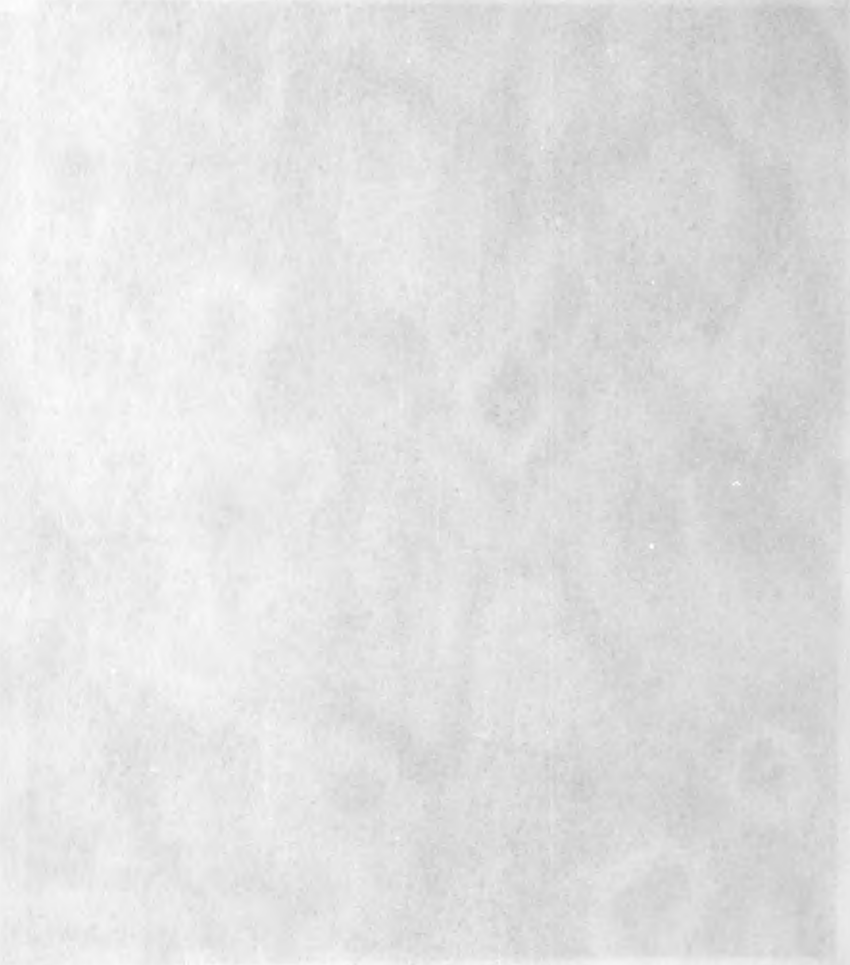


FIGURE 9



JOURNAL

POWER DIVISION

Proceedings of the American Society of Civil Engineers

# WATER RESOURCES AND POWER STUDIES, TASK FORCE, HOOVER COMMISSION: ORGANIZATION AND SCOPE: CONCLUSIONS AND RECOMMENDATIONS[1]

Ben Moreel,[2] Hon. M. ASCE

## FOREWORD

This paper is one of a group of five concerned with various phases of the work of the Water Resources and Power Task Force of the Hoover Commission. All were presented at the June 15, 1955, meeting of the ASCE in St. Louis.

The specific function of the Hoover Commission was to study and report to Congress on ways and means of promoting economy, efficiency and improved service in the transaction of the public business in the Executive Branch of the Government. Work of the Commission was implemented by thirteen Task Forces, one of which was the Task Force on Water Resources and Power. Work of this Task Force was divided among four Task Groups assigned the following fields:

1) Power generation and distribution
2) Reclamation and water supply
3) Flood control
4) Navigation

The five papers presented include four by the Chairmen of these Task Groups covering work of their particular groups and one by Admiral Moreell, Chairman of the Task Force. The first of these papers, "Organization and Scope; Conclusions and Recommendations," (Proc. Paper 897) by Ben Moreell, Hon, M. ASCE, explains the organization and scope of the Task Force and presents his personal conclusions and recommendations. John Jirgal in his paper, "Power Generation and Distribution," (Proc. Paper 898) cites the fact that in 1933 the Federal Government owned less than 1% of the total electric generating capacity in the United States; in 1953 the proportion was 12 1/2% and in 1960, at the present rate, it will be 16%. The necessity

Note: Discussion open until June 1, 1956. Paper 897 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, PO 1, February, 1956.

1. Paper presented at St. Louis meeting of American Society of Civil Engineers, Power Division, June 15, 1955.
2. Chairman of the Board, Jones & Laughlin Steel Corp., Pittsburgh, Pa.

of conserving the water supply and developing an overall Federal water policy is stressed in "Reclamation and Water Supply" (Proc. Paper 899) by Leslie A. Miller. In "Flood Control" (Proc. Paper 900) W. W. Horner, M. ASCE, reviews the number and cost of presently constructed or authorized, and future probable, flood-control projects. "Improvements to Navigation" (Proc. Paper 901) by Carey H. Brown, M. ASCE, is the last of the papers; it is concluded therein that many navigational improvements now under consideration or recommended are of dubious economic value.

Since the Task Force Report had not yet been released the authors were unable to summarize the report itself or to disclose its conclusions and recommendations. They were, however, able to outline in these papers facts found in the public records and problems presented by these facts. Necessarily all opinions and conclusions are those of the individual authors.

## SYNOPSIS

The Task Force on Water Resources and Power, one of thirteen such groups in the second Hoover Commission, was formed in November 1953. The Task Force was divided into four Task Groups charged with studying these principal areas:

1) Power generation and distribution, including atomic power.
2) Reclamation and water supply including domestic and industrial water supply, irrigation, waterway pollution, recreational projects, fish and wildlife projects and drainage.
3) Flood control, including water retardation projects and upstream watershed treatment.
4) Improvements to water navigation, including inland water transportation and beach erosion.

Members of the Task Force were chosen from geographically dispersed areas and were all prominent men in their respective communities and in the top ranks of their respective professions. No representatives of either public or investor-owned power or other interests were on the Task Force.

Public hearings were held in San Francisco, Denver, New York, Chattanooga and Portland, Oregon, and closed hearings were held in Washington with the Federal Power Commission, the Department of Agriculture, the Department of Interior and the Corps of Engineers. Some two hundred projects were studied in detail.

Basic considerations which guided the Task Force in its studies were (a) the total of Federal funds invested to date, and (b) the rapid increase in the rate of growth of Government expenditures for water resource development.

In 1954 there were 43 offices and Bureaus of Federal Government involved in water resources and power development. Of these, 25 are concerned with this work as a primary responsibility or as a major activity. Their 1954 expenditures approximated $1.5 billion and more than 70,000 employees were on their payroll.

It is concluded that Federal activities in the fields of water resource and power development have taken place without central supervision by the Executive Branch of the Government, except for cursory action by the Bureau of the Budget, which has never been adequately staffed for this purpose.

Furthermore, there is no clear-cut unified body of Federal water policy. Instead there are diverse policies for different types of development and different agencies, variously authorized and financed, each programmed under different ground rules and often in conflict and competing with one another.

* * * * *

## Part I - Scope and Organization

It is regretted that the Report of the Hoover Commission on the work of our Task Force on Water Resources and Power is not yet released. We cannot divulge the Task Force Report contents until the Commission has released it. Therefore, the statements made in this discussion are those of the writer and may, or may not, agree with the Task Force conclusions and recommendations. However, with respect to statements of fact, these are supported by official data in the Task Force Report.

The second Hoover Commission was set up by Act of Congress approved July 10, 1953, and was organized in September of that year. It expired June 30, 1955.

The Commission was authorized to study and report to the Congress on ways and means of promoting economy, efficiency and improved service in the transaction of the public business in the Executive Branch of the Government. The Commission was prohibited from investigating the two other coordinate Branches of Government, namely, the Legislature and the Judiciary.

The scope of the second Hoover Commission differed materially from that of the first. The latter was limited to studies and recommendations with respect to the performance of the then existing functions of Government. The second Commission, in addition to the duties of its predecessor, was authorized to make studies involving matters of policy, that is, to determine not only whether an existing function is being performed efficiently but, also, whether it should be done by Government at all, and if so, to what extent and in what manner.

While everything having to do with Government involves political considerations, more or less, it is obvious that the first Commission was a far lesser political irritant than the second. The first was concerned primarily with the shrinkage, re-shuffling and consolidation of agencies. This resulted in loss of jobs, and thus caused some political pain. The second, however, was authorized, also, to recommend the elimination of functions. This involves not only abolishing jobs but, also, the reduction or cancellation of subsidies and of outright grants. It is certain that when such measures are proposed, the voice of the anguished will again be heard through the land, crying, "Don't let them take it away."

The second Hoover Commission carried on its studies by means of thirteen Task Forces and a Committee on Business Organization of the Department of Defense. The Task Force on Water Resources and Power was organized in November, 1953. In addition to the Chairman, there are twenty-five members.

When pet projects are attacked, the time-honored tactic of demagogues and pleaders for special privilege is to assail the integrity and competence of the attackers. Since our Task Force has been thus assailed, even before we were well started on our work, and, more recently, as a result of "leaks" of

the Task Force Report, I want to make a statement with respect to the criteria used in selecting members of this Task Force.

First, it was agreed that we should have geographical dispersion of membership; also, that there should be no representatives of either public or investor-owned power or other interests. It was stipulated that all should be prominent members of their respective communities, men of unassailable integrity who command public respect; and that they should be in the top ranks of their respective professions. No inquiry was made as to their political affiliations and to this day I do not know what they are. Nor was an inquiry made as to their views on the controversial matters we were to investigate, it being assumed that men of this caliber would arrive at honest conclusions once all the facts were known.

However, they were asked whether they were in sympathy with the mandate of the Commission, which is to recommend changes in organization and operations of the various government agencies as may be necessary to promote economy, efficiency and improved service in their transaction of the public business. An affirmative reply was required.

The Task Force members were recruited from twenty different States of the Union. Three came from the Northeast; five from the Middle Atlantic; two from the South Atlantic, three from the Central States; four from the Pacific Coast; four from the Mountain States; one from the Plains States; and two from the Gulf Southwest area.

Professionally, fourteen are engineers; six are lawyers; two newspaper publishers; three present or former state governors; four business executives; one consultant in utility economics and finance; and one accountant. This totals more than twenty-six because some members fall in more than one category. Two are former presidents of the National Reclamation Association; and three have served as officers in the Corps of Engineers of the Army. A number of the members have worked closely with the Corps of Engineers and the U. S. Reclamation Service and several were at one time employed by the Reclamation Service. A rough calculation indicates that, as a whole, the Task Force members have spent considerably more time working for public agencies than for private industry.

Eleven of the Task Force members are members of the American Society of Civil Engineers; two of these, Messrs. W. W. Horner and Malcolm Pirnie, are past presidents. Mr. W. D. Shannon is a past vice president and Messrs. Julian Hinds and Royce J. Tipton are past directors. There are six consultants to the Task Force of whom two are members of the Society. Of the forty-one people who, at various times, served on the staff of the Task Force, eight are members of the Society, including one of the present vice presidents, Mr. Frank L. Weaver.

The Task Force was charged with making studies to be used as a basis for conclusions and recommendations which would enable the Commission to carry out its mission in the field of water resources and power. The scope of the Task Force work was defined as covering four principal areas as follows:

1) Power generation and distribution, including atomic power.
2) Reclamation and water supply, including domestic and industrial water supply, irrigation, waterway pollution, recreational projects, fish and wildlife projects, and drainage.
3) Flood control, including water retardation projects and upstream watershed treatment.

4) Improvements to water navigation, including inland water transportation and beach erosion.

It was obvious that, in order to cover a field of such large scope and complexity, a formalized organization and procedures of corresponding scope and detail would be required. There was set up, under the Chairman, a traditional "line and staff" organization. This is shown on the accompanying chart. The line organization consists of four Task Groups with the jurisdiction I have stated. The staff organization consists of (1) a group of six consultants, (2) the administrative staff, under the direction of the Administrator, Charles D. Curran, (3) an advisory board to the Chairman, (4) an accounting advisor and (5) a general counsel.

The administrative staff, under Mr. Curran, was organized into legal, engineering, accounting, editorial, statistical and clerical groups.

In addition to the organization shown on the chart, special consultants were engaged on a temporary basis to make studies covering important aspects of water resources and power developments. These consultants functioned under the Administrator.

Not shown on the chart are two informal sub-committees, one known as the "Program Group," consisting of the Chairman of the Task Force, three consultants, and the Administrator; and the other, "The Planning and Coordinating Committee," consisting of the Chairman of the Task Force, the four Chairmen of the Task Groups, the accounting adviser and the general counsel. The Program Group acted as a "Steering Committee" to set up progress schedules and agenda for meetings. The Planning and Coordinating Committee set up standards for procedures, defined the limits of the work of each of the Task Groups, prevented duplications and over-lapping and coordinated the work.

Because of the limited time available and the large scope of the work, special efforts were made to recruit a staff of high caliber. For the senior staff, we obtained professionals of broad experience. Most of these were borrowed from the Federal Agencies. Emphasis was placed on finding men of objective judgment, not committed to doctrinaire views, who were able and willing to subject policies and organization to critical analysis. I must here pay tribute to the Federal Agencies who assisted us and who were very generous in providing personnel and data.

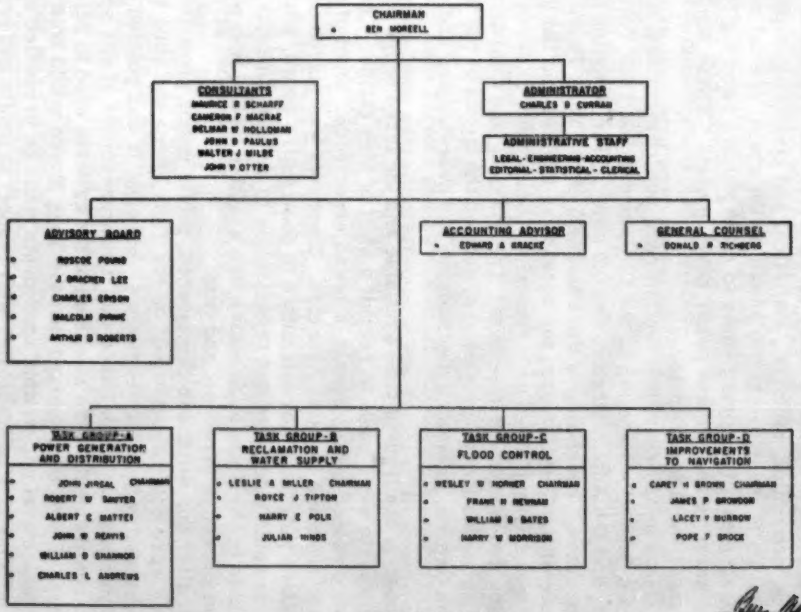
Soon after our work was commenced, public power proponents complained that they were not "represented" on the Task Force and such representation was demanded with great vehemence. In reply, it was pointed out that none of the members of the Task Force was a representative of any special interest and the complainants were informed of the criteria used to select Task Force members. It was then decided that the Task Force would hold public hearings at which oral and written statements could be presented by the advocates of any policies and/or ideologies with respect to public and private ownership and operation, subsidies, taxes, jurisdiction, organization, etc.

The Task Force held public hearings in five cities - San Francisco, Denver, New York, Chattanooga and Portland, Oregon, - at which 261 witnesses presented statements. The testimony covers in excess of 5,000 typewritten pages. It has not been printed but is available to the public for inspection at the Commission offices.

In addition to the public hearings, closed hearings were held in Washington with the Federal Power Commission, the Department of Agriculture, the

COMMISSION ON ORGANIZATION OF THE EXECUTIVE BRANCH OF THE GOVERNMENT

# TASK FORCE ON WATER RESOURCES AND POWER




* MEMBERS OF TASK FORCE

Ben Morell
CHAIRMAN

REVISED - MAY 1, 1956

Department of Interior and the Corps of Engineers. Also, a closed hearing was held for Senator Watkins of Utah, who was accompanied by representatives of the Bureau of Reclamation, and certain officials of the State of Utah who wished to present data in support of the Upper Colorado River Storage Project.

The Governors of the States and Territories also were invited to present their views and they responded with seventy-eight written statements.

Twelve major field inspection meetings were held with the Federal Agencies and a number of special trips to field projects were made by individual Task Force members, by the consultants, and by staff members.

With the cooperation of the Federal Agencies concerned, some two hundred projects were studied in detail. These included every reclamation project and every federal power system. Important examples of flood control and navigation projects were also studied.

In the official instructions issued to the Task Force on November 6, 1953, there was included this sentence: "Every member of the Task Force will have complete freedom to state his own opinion on all matters under consideration." It was my intention to avoid any implication of "steam rollering," so that the finished work of the Task Force would represent a reasonable consensus. I felt that to obtain unanimity on all matters under consideration from twenty-six men of such diverse backgrounds was impossible and that the value of the Report would be enhanced if dissenting opinions were published. I do not imply that the absence of dissents indicates that all members are in complete agreement with the views expressed. No doubt there are some disagreements but where these were not submitted for publication it is to be assumed that the disagreements were considered not of sufficient importance to warrant a formal dissenting statement.

Only three such statements were presented for publication, while two additional dissents were registered, not for publication but merely "for the record." All dissents referred to specific matters of limited scope, not to the report as a whole.

In addition to the public and closed hearings mentioned above, there were, between November 4, 1953, and March 15, 1955, eighty-one meetings of the Task Force, Task Groups, Program Group and the Planning and Coordinating Committee.

The Task Force Report was submitted to the Commission on March 31, 1955, and is now being printed. It will consist of three volumes. The first volume is the Report of the Task Force proper and consists of approximately 270 pages of closely printed matter. The second volume consists of the Reports of the four Task Groups, approximately 730 pages, and the third volume of 780 pages is a compilation of the special studies prepared by the staff members and the special consultants. The total report will, therefore, have about 1,780 pages.

Volume I, the Task Force Report proper, consists of five chapters, as follows:

Chapter I - Basic Considerations and Findings
Chapter II - Policies Past, Present and Future
Chapter III - Federal Executive Organization, Present and Future
Chapter IV - Recommendations
Chapter V - Examples, Facts, and Analyses Behind the Recommendations

Volume I also contains the dissents and appendices with data relative to administrative procedures of the Task Force.

Below is given a birds-eye view of some of the basic considerations which guided the Task Force in its studies. Chief among these is the importance of the subject in terms of (a) the total of Federal funds invested to date, and (b) the rapid increase in the rate of growth of government expenditures for water resources and power at a time when Federal budgetary deficits have become chronic and the Federal debt is at an all-time peak.

Since 1824, when the Federal Government first initiated work on the water resources of the country, a total of $14.3 billion has been spent. Of this, $11.6 billion has been capital outlay and the remainder for maintenance, operation and planning. Projects already authorized call for an additional capital outlay, estimated by the Federal Agencies concerned at $18-1/2 billion. This authorized backlog thus calls for an expenditure over 50 per cent greater than the total expenditure during the past 131 years.

But this is not all. In 1938, the Corps of Engineers made an estimate of the cost of water resources projects then contemplated and came up with a figure of $57-1/2 billion at 1948 price levels. This includes $29 billion for projects not authorized at that time. Since then, the 1950 and 1954 Rivers and Harbors Flood Control Acts have been passed and numerous reclamation projects and the St. Lawrence River Improvement Project have been authorized. The $57-1/2 billion, mentioned above, included only $4 billion for upstream watershed treatment. Under the 1954 Act providing for such projects, our staff estimated that this program will amount to $17 billion or more. Thus, a very conservative estimate of the backlog now authorized, or contemplated, by the Federal Agencies is in excess of $70 billion. This is almost seven times as much as was spent in the first century and a quarter of these activities. With respect to the rate of expenditures, it is significant to note that up to 1920 only 8 per cent of the present $14.3 billion total had been spent. Between 1920 and 1930, 6 per cent was spent, and since 1930, 86 per cent of the spending has taken place, with the bulk of it (68 per cent) crowded into the post-was years. Between 1940 and 1950, $5.2 billion were spent and from 1950 to 1954, $4.3 billion.

In 1954 there were 43 offices and Bureaus of the Federal Government involved in water resources and power developments. Of these, 25 are concerned with this work as a primary responsibility or as a major activity. Their 1954 expenditures approximated $1.5 billion and more than 70,000 employees were on their payrolls.

From these figures, and having in mind the rapidly mounting public debt of the Federal Government, it is evident that a critical examination of the program is long overdue.

There seems to have developed in this country a belief that the expenditure of Federal funds to promote the interests of a favored area, or of a favored group, is warranted, provided it can be shown that the <u>probable</u> resulting benefits to the National economy exceed the initial estimated out-of-pocket costs. Parenthetically, it is traditional that benefits are largely overestimated and costs underestimated. The question whether this is the <u>best</u> expenditure of Federal funds in the interest of all of the people of the Nation, who pay the bill, seems to have been neglected or, if not neglected, is passed over lightly. And I am sure that anyone who would have the temerity to raise a question as to the morality of taking money from all the people to benefit a favored few would run the risk of being branded a "crackpot," or, at the very

least, a black reactionary who wants to "turn back the clock to the horse and buggy days."

Man's wants are insatiable. Given freedom of choice, an individual will trade the things he has for other things if he believes the latter will give him greater satisfactions. If he has money, he may choose to exchange it for certain things from which he will derive the maximum satisfactions per dollar of expenditure. Or he may choose to keep it. But in the field of public finance and public welfare it would appear that this criterion does not apply and that we consider only the question - Will it do good? If the answer is "yes," we proceed with the project, presumably on the theory that the public treasury has "no bottom."

These "benefactors with other people's money" set themselves up as judges to determine, for the rest of us, which satisfactions would be most rewarding to us! From an examination of the record I am convinced that in some cases, at least, their judgments have not been without an element of self-interest. There is ample evidence that political motives have frequently weighed heavily in the determination of whether the government should or should not build.

In the foregoing I have tried to outline the mission and organization of our Task Force. In the following I shall attempt a summary statement of some conclusions which might be drawn from the Task Force Studies and the statements of the Group Chairmen, and how I think the recommendations of the Hoover Commission to the Congress could be implemented.

## Part II - Conclusions and Recommendations

I believe the most important conclusion is that Federal activities in the fields of water resource and power development have taken place without central supervision in the Executive Branch of the Government, except for cursory action by the Bureau of the Budget, which has never been adequately staffed for this purpose. Furthermore, there is no clear-cut, unified body of Federal water policy. Instead there are diverse policies for different types of development and different agencies, variously authorized and financed, each programmed under different ground rules and often in conflict and competing with one another.

There is an imperative need for a clear-cut definition of the role and policies of the Federal Government within the framework of a consistent National water policy. The current Federal policies taken together result in a hodge-podge, many elements of which are contradictory. This, in turn, generates conflict among agencies, over-lapping of functions, competition for position, and wasteful expenditures.

In keeping with the growth trend of all levels of government—Federal, State, county and municipal—together with the policy over the past few decades of concentrating more and more power in Washington, the Federal Government has steadily assumed a larger and larger share of responsibility for water resource and power development. So much so that it has now become a dominant factor in enterprises which, under our traditional concept of sovereign states, should be outside its domain.

In the field of hydroelectric development, the Federal Government has invaded a field which should be reserved for private enterprise. In 1933, the installed capacity of Federal power projects was less than one per cent of

all electric utilities of the country. By 1953, the Federal Government had become the largest single producer of electric power, producing 13.1 per cent of all kilowatt-hours generated. Technically and financially, there is no present or prospective need for Federal power activities. It is logical to conclude that no additional Federal power projects should be constructed and that a start should be made on disposing of Federal projects to private industry or, if this is not feasible, to the States and local governments.

Federal power projects under construction and authorized (if these be also constructed) will, by 1960, more than double the 1953 capacity and, when all are completed, will triple it, reaching a total of 35 million kilowatts of installed capacity and representing a total outlay of $10 billion.

There is no lack of ability on the part of privately-owned utilities to finance and install needed electric generating, transmission and distribution facilities. Since the end of World War II, they invested approximately $20.0 billion in electric properties as against an approximate total expenditure by the Federal Government from the very beginning of its power activities of a mere $2.5 billion! And much more is now available for financing sound private projects. The Federal Government does not owe a responsibility to supply any community, section or region with its power requirements.

Another indication of the powerful movement toward centralized government is that the Federal Government has not, in the recent past, given sufficient consideration to non-Federal interest, opinion and participation in water resource and power projects and there is every indication that local desires will receive less consideration in the future, unless this trend is stopped.

In the last two decades, the Federal Government has shown a strong tendency to designate whole valleys and river basins as exclusive Federal domains for development of water resource projects instead of encouraging local and private interests to undertake these works.

At the public hearings held by the Task Force, state highway officials again and again called attention to the fact that decisions on bridge clearances across navigable streams and whether or not draw bridges should be installed are made without proper attention to the extra cost to State highway departments or delays to highway traffic and that, frequently, the water traffic so accommodated is minor to non-existent.

While some current legislation favors a closer relationship between State and Federal interests, it is noteworthy that Federal agencies do not generally consult effectively either among themselves or with State and local groups. Unfortunately, local authorities, in general, will not question projects which are paid for with Federal funds, preferring to yield to the powerful seduction of subsidies. In this case, as in many others, "he who pays the piper calls the tune"; the Federal Government bosses the show.

It is apparent from our studies that the Federal Government has used water resource and power development projects, which should be undertaken exclusively for economic purposes, to accomplish indirect social and political ends. There are instances where projects are described as flood control, reclamation or navigation measures when their real and obvious purpose is to produce hydroelectric power. Navigation improvement projects have been constructed solely in order to bring pressure to bear on the railroads and truckers to reduce freight rates. Although Federal power production is described by existing legislation to be subservient and supplemental to other objectives of water resource development, Federal agencies have

recommended construction of multiple-purpose projects which were but thinly disguised hydroelectric developments. Also, Federal projects have been recommended whose sole purpose is the development of power with no attempt at disguise.

The construction of Federal power projects as a so-called "yardstick" to control the rates charged by investor-owned utilities is a snare and a delusion. I was taught in my school days here in St. Louis that there are 36 inches to the yard. But by virtue of failure to include taxes and because of low-interest financing, as well as other hidder subsidies, the yardstick proposed by public power proponents has somewhere in the neighborhood of 23 inches.

The improvement, in recent years, of regulatory techniques and the successful administration of the various controls instituted by the States and the Federal Government over both operations and investments have been shown to provide ample safeguards for both consumer and investor. No "short-change yardsticks" are required.

Another conclusion which might be drawn from our studies is that the Federal Government has paid too much of the costs of water resource and power development and has required too little of the beneficiaries. Of the total of $85 billion invested by all public and private sources in water resource development in the United States, the Federal Government has made a capital outlay of about $12 billion. The other $73 billion has been spent without benefit of Federal subsidy and, thus, the persons who financed those expenditures have been discriminated against, as have the other taxpayers of the country.

In some cases, beneficiaries have contributed a share to the costs of Federal projects but, with the exception of some unusual cases, the contribution has been minor.

In recent irrigation projects, the general range of repayments required from the beneficiaries has been between 1/4 and 1/3 of the capital costs and a few are as low as 10 per cent. In no case have they been required to pay interest.

The Federal Government usually subsidizes completely whatever benefits occur in government projects classed as recreational, or for fish and wildlife, or pollution abatement.

With respect to navigation and certain types of flood control, local contributions average only a few per cent of Federal expenditures. No contribution is required from local sources for flood control dams and storage reservoirs.

A study was made of a large sample of flood control projects representing 75 per cent of cost of $1.4 billion and listed as currently active by the Corps of Engineers. The check showed that for active projects in the categories that call for local contributions, the average of such contributions was about 12.6 per cent of total costs. For <u>all</u> active flood control projects of all kinds, a total of $3.18 billion, the local contributions total 6.1 per cent.

For navigation programs, cash contributions of local beneficiaries total about two per cent of the total costs of projects for which the Federal Government appropriated $2.5 billion.

An analysis of watershed flood control projects of the Department of Agriculture shows that for a total program of 212 completed dams, the Federal investment will total 98.8 per cent of construction costs.

From an analysis of the projects studied by the Task Force it is fair to conclude that the Federal Government has planned, constructed and paid for

water resource and power development projects which are economically unsound and, hence, waste the National wealth. It is quite evident that project costs are frequently underestimated, that benefits advanced by various Federal agencies as justification for projects are often exaggerated, that unsupported and unsupportable claims are made for indirect benefits, and that benefits produced by economically sound features are used to compensate for uneconomical features, either by combining purposes of multiple-purpose structures or by combining projects, or both, in a single economic evaluation. In some instances, projects have been recommended and authorized without economic justification of any kind.

Flagrant examples of underestimation of costs are cited in the Task Force Report. But this is only part of the problem. Overestimating benefits, which is less easy to detect, is the other. The evaluation of so-called "indirect benefits" can, in many instances, be properly described as pure fantasy. When the skilled evaluator of secondary benefits hits his stride the sky is the limit, and he finds no difficulty in arriving at evaluations for mental, moral, physical, intellectual, psychic and social satisfactions which will be derived by individuals and groups from the project he is justifying, nor does he have any difficulty in assigning specific dollar values to each of these so-called benefits.

The Task Force Report cites chapter and verse of flagrant examples of such overestimations of benefits. I shall not attempt to repeat them here. However, there is ample evidence to warrant the conclusions that the desire of rival Federal agencies to enhance their prestige by construction of large projects, plus the fact that Congressional Committees often accept the agencies' economic evaluations without thorough scrutiny, have resulted in the construction of projects which are of little worth to the Nation and, in some instances, even to the local communities.

Another conclusion which might well be drawn from the data compiled by the Task Groups is that Federal water resource and power projects which produce, or could produce, revenues are not operated according to sound business principles and do not produce a return fairly related to their value.

Even though capable of fully paying their way, all Federal power projects now contain some subsidy which is paid for by the taxpayers of the country and redounds solely to the benefit of local consumers.

Similarly for other water resource projects, such as flood damage abatement, river improvements and reclamation. All beneficiaries should make contributions commensurate with the benefits they receive. There is no other way to achieve an equitable adjustment between benefits received by the few and costs paid by the many.

The Task Force devoted considerable time to the study of the Federal Government's organization for carrying our water resource and power projects. There is every indication that there is a lack of coordination which fosters competition among the agencies, causes controversy, confusion, duplication and waste, and encourages bureaucratic ambitions. Nor is there at present any provision for effective procedures for accomplishing an independent and objective review of water resource and power projects which are proposed by the various agencies.

It is quite obvious that the two most important organizational matters now are—first, effective coordination and, second, independent review. The Task Force has made specific recommendations relative to both of these matters.

It is obviously impossible to effectively condense the content of some 1,800 pages of data in a brief commentary of this kind. I recommend to those who are interested in this vital subject that they obtain a copy of the Task Force Report, from which they will be able to cull detailed supporting data on the findings and recommendations cited by the Task Force.

The last part of my assignment has to do with the methods of implementing the Hoover Commission recommendations to the Congress.

Because of the reluctance of political administrators to deprive anyone of a favor which he now enjoys, it is my firm conviction that the only way the reform measures recommended by the Hoover Commission can be made effective is for the citizens of this country to get behind them and to make their voices heard in the halls of Congress, through the medium of their elected representatives.

Many years ago, I was told by an old-time member of the United States House of Representatives that his formula for a successful political career was to vote for every appropriation measure and against every tax measure. Obviously, his decisions were based on purely political judgments. When purely economic judgments are to be made, it appears that it will be necessary for the electorate to make them and to become vocal in their insistence that action be taken by their legislative representatives.

I am sure that among the Hoover Commission recommendations some will be found which will call for disagreement and which would not win general support. This is inevitable. The Hoover Commission is a bi-partisan organization, designed to represent both political parties. I believe it is fair to state that, in many instances, the Commission's recommendations represent compromises between sound economic judgments and political expediencies. When such compromises are made, the end result is usually not entirely satisfactory to either party. I am sure that some of the Commission recommendations with respect to the work of our Task Force will be such that I cannot support them. I am equally sure that, on balance, the total of the Hoover Commission recommendations, if made effective, would constitute a great advantage to the Nation.

It is my firm intention to do all in my power to mobilize effective public support for those recommendations of the Hoover Commission with which I agree. If this were done by all citizens, the result would be reflected in remedial action by the Congress. I, therefore, urge careful consideration of all recommendations and support for those which appear acceptable.

JOURNAL

POWER DIVISION

Proceedings of the American Society of Civil Engineers

# WATER RESOURCES AND POWER STUDIES, TASK FORCE, HOOVER COMMISSION: POWER GENERATION AND DISTRIBUTION[1]

John Jirgal[2]
(Proc. Paper 898)

## FOREWORD

This paper is one of a group of five concerned with various phases of the work of the Water Resources and Power Task Force of the Hoover Commission. All were presented at the June 15, 1955, meeting of the ASCE in St. Louis.

The specific function of the Hoover Commission was to study and report to Congress on ways and means of promoting economy, efficiency and improved service in the transaction of the public business in the Executive Branch of the Government. Work of the Commission was implemented by thirteen Task Forces, one of which was the Task Force on Water Resources and Power. Work of this Task Force was divided among four Task Groups assigned the following fields:

1) Power generation and distribution
2) Reclamation and water supply
3) Flood Control
4) Navigation

The five papers presented include four by the Chairman of these Task Groups covering work of their particular groups and one by Admiral Moreell, Chairman of the Task Force. The first of these papers, "Organization and Scope; Conclusions and Recommendations," (Proc. Paper 897) by Ben Moreell, Hon, M. ASCE, explains the organization and scope of the Task Force and presents his personal conclusions and recommendations. John Jirgal in his paper, "Power Generation and Distribution," (Proc. Paper 898) cites the fact that in 1933 the Federal Government owned less than 1% of the total electric generating capacity in the United States; in 1953 the proportion was 12-1/2% and in 1960, at the present rate, it will be 16%. The necessity of conserving the water supply and developing an overall Federal water policy

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1. Paper presented at St. Louis meeting of American Society of Civil Engineers, Power Division, June 15, 1955.
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is stressed in "Reclamation and Water Supply" (Proc. Paper 899) by Leslie A. Miller. In "Flood Control" (Proc. Paper 900) W. W. Horner, M. ASCE, reviews the number and cost of presently constructed or authorized, and future probable, flood-control projects. "Improvements to Navigation" (Proc. Paper 901) by Carey H. Brown, M. ASCE, is the last of the papers; it is concluded therein that many navigational improvements now under consideration or recommended are of dubious economic value.

Since the Task Force Report had not yet been released, the authors were unable to summarize the report itself or to disclose its conclusions and recommendations. They were, however, able to outline in these papers facts found in the public records and problems presented by these facts. Necessarily all opinions and conclusions are those of the individual authors.

## SYNOPSIS

The generation and sale of electric energy is the Federal Government's largest and most rapidly expanding business. In 1933, the Federal Government owned and operated less than one per cent of the total utility plant capacity in the United States. By 1953 its proportion had increased to about 12-1/2 per cent and by 1960 it will be about 16 per cent. At the end of 1953 Federal generating plants had a capacity of 11.5 million kilowatts. These utility plants and their transmission systems cost Federal taxpayers $2.3 billion. Upon completion of the plants now under construction and those authorized, total Federal generating capacity will be close to 35 million kilowatts costing almost $10 billion.

Ninety per cent of entire Federal power capacity and generation is concentrated in five river basins, namely: the Tennessee River Valley, Columbia River Valley, Colorado River Valley, the Central Valley Project in California and the Missouri River Valley projects.

Although engineering work and physical operation are in keeping with the best practices of the utility field, these are only a small part of overall economy and efficiency in any power business. A well-designed plant, competently operated, will prove uneconomical and inefficient if the power it generates is not or cannot be sold at prices which cover full costs of operation, including a fair return on investment. Two main Federal policies run directly contrary to that principle, namely;

1) That the power be sold at lowest possible rate which would promote wide-spread use, and
2) That preference in disposition of power be given to states, municipalities, rural cooperatives and public bodies, generally.

It is estimated that in 1953 the Government sold power that it generated for $110 million to $130 million less than it was really worth and when presently contemplated projects are completed, this annual subsidy to consumers of Federal power could amount to $400 million per year.

* * * * *

The subject of this paper is the Federal Government's largest and most rapidly expanding business venture—the generation and sale of electric energy.

The writer was Chairman of the Task Group on Power Generation and Distribution. The other members of this Group were C. L. Andrews, of the C. L. Andrews Cotton Co., of Memphis, Tennessee; A. C. Mattei, president of the Honolulu Oil Corp., of San Francisco, California; J. W. Reavis, an attorney of Jones, Day, Cockley and Reavis, Cleveland, Ohio; Judge Robert W. Sawyer, former president of the National Reclamation Association and former editor and publisher of Bend, Oregon; and W. D. Shannon, consulting engineer of Seattle, Washington.

As you will note, this is a representative group of business and professional men, none of whom is connected with the operation or management of an electric utility either privately or publicly owned.

We were assigned the job of examining the Federal Government's power activities in the light of the aims set out by Congress in establishing the Hoover Commission. These aims were to see what could be done to promote economy, efficiency and improved service in the public business. We were to consider particularly if any power activities not necessary to the efficient conduct of the Government could be abolished and if any non-essential activities in this field are competitive with private enterprise and could be eliminated for that reason.

In view of the scope of the Federal Government's power activities, our task involved an enormous amount of work. We were able to complete our extensive investigation only because of the able and conscientious efforts of our hard-working staff, headed by Frank L. Weaver, ASCE vice president, and George G. Adkins, both high ranking engineers loaned to us by the Federal Power Commission. We also had the benefit of engineering advice from Colonel A. B. Roberts of Cleveland, Ohio, a member of the Task Force's Advisory Committee and consultant to the first Hoover Commission, and of the valuable consulting services of Walter J. Milde, an attorney of Jones, Day, Cockley and Reavis, Cleveland, Ohio.

The results of our studies and the recommendations we made were set forth in a three hundred page report to Admiral Moreell's Task Force and transmitted as a part of the Task Force Report to the Hoover Commission. Since the Commission has not yet acted, and the various reports to it have not been published, I am not free to summarize our report for you but I can state the facts we found in the public records and outline the problems these facts present. I will also tell you what I, as an individual, think about the solution to some of these problems.

In 1933 the Federal Government owned and operated less than one per cent of the total utility plant capacity in the United States. By 1953 its proportion had increased to about 12-1/2 per cent and by 1960 it will be about 16 per cent. About 78 per cent of the total utility capacity was in private utility hands in 1953 and somewhat less than 10 per cent was owned by non-Federal public bodies, including cooperatives.

At the end of 1953 Federal generating plants had a capacity of about 11.5 million kilowatts. These utility plants and their transmission systems cost the Federal taxpayers $2.3 billion. Upon completion of the Federal plants now under construction and presently authorized, the total Federal generating capacity will be close to 35 million kilowatts—three times the 1953 capacity. They will have cost the Federal taxpayers almost $10 billion to construct. There are further projects which have been proposed but not authorized which, if undertaken, will run into additional millions of kilowatts and billions of dollars.

The Tennessee Valley Authority alone operates the largest power generating system in the United States. On the basis of present plans TVA will, by 1960, have an estimated 8 per cent of all of the utility plant capacity in the United States. Its power investment, including transmission, will be over $3 billion when projects scheduled to commence prior to 1960 are completed. Its generating capacity will then be over 13-1/2 million kilowatts. About 75 per cent of that capacity will be steam-electric. TVA deliveries to the Atomic Energy Commission will shortly be about 50 per cent of its generation but even without this load TVA will, on the basis of present plans, operate the largest power generating and transmission system in this country for many years to come.

The Columbia River Basin power properties of the Federal Government make up the second largest Federal system. It also will have an investment of over $3 billion when the plants under construction and authorized have been completed. Its generating capacity will then be over 11 million kilowatts.

These two Federal systems alone will have over 70 per cent of the capacity and over 65 per cent of the investment of the entire Federal power undertaking.

The other large Federal systems are the Colorado River development, which includes the Hoover, Parker and Davis dams, the Central Valley Project in California and the Missouri River Valley projects. Together these five Federal systems will, when the presently authorized construction is completed, make up about 90 per cent of the entire Federal power capacity and generation.

Let it be noted that these five Federal power systems are in a few river basins. Hydroelectric power is unequally distributed throughout the country. Over 60 per cent is in the Mountain and Pacific states; over 40 per cent is in the Pacific Northwest states of Washington, Oregon, Idaho and Montana. There are no Federal power plants in New England or the middle Atlantic states, and none of importance in the east north central states. As an example of this unequal distribution, the states of New York, New Jersey and Pennsylvania, which do not have a single Federal power project, have only 7 per cent of the nation's potential hydroelectric power but account for 29 per cent of the Federal income taxes paid, have 20 per cent of the population and presently use 19 per cent of all the electricity sold.

The billions invested in Federal power to which I have referred do not include the $2.7 billion of Federal loans approved by the Rural Electrification Administration. These additional billions have furnished over 90 per cent of the investment in rural lines, power plants and transmission systems made by REA's borrowers.

In view of this large investment it is obvious that Federal power activities are big business. Actually the Federal taxpayers will shortly after 1960 have invested in this large scale commercial undertaking nearly as much money as security holders have invested in the American Telephone and Telegraph Company system and from twice to several times as much as they have invested in General Motors Corporation, Standard Oil Company of New Jersey, and DuPont chemical undertaking or the Pennsylvania Railroad. The Federal investment will then also be several times that of any of the larger privately owned electric systems.

Is this Federal power business being run economically and efficiently? Is all of this business necessary? Is any of this business a non-essential activity competitive with private enterprise?

First, let me take up economy and efficiency.

As to engineering work and physical operations it appeared to our Group that these were in keeping with the best practices followed by the electric industry generally. We were impressed with the competence of the present field supervisory forces with which we came in contact.

But sound construction engineering and proper physical operations are only a small part of overall economy and efficiency in any power business. A well designed plant, competently operated, will prove uneconomical and inefficient if the power it generates and transmits is not or cannot be sold at prices which cover the full costs of operation including a fair return on the investment.

Most Federally generated power is marketed by the Department of Interior through four agencies; The Bureau of Reclamation, which disposes of the power generated at the Hoover-Parker-Davis dams, at the Central Valley project in California and at a number of the smaller and more isolated plants connected with irrigation projects; The Bonneville Power Administration, which sells the Columbia River basin output; and the Southeastern and Southwestern Power Administrations which market the generation of certain power plants built in connection with flood control projects in the Southeastern and Southwestern portions of the United States. The TVA markets its own power directly.

Most of these Federal power generating plants were constructed as a part of multiple-purpose projects, the primary object, either expressed or implied, being to improve navigation, prevent floods or provide irrigation, or some combination of these. Power development was to be incidental to these main objectives. Since the greater share of the cost of the other phases of these projects were considered to be non-reimbursible, it would seem logical to have sold this surplus product for what it was fairly worth. In this way, the earnings from power sales would have reduced, to the maximum extent, the cost of the entire project and thus relieve the Federal taxpayers of some part of their burden.

Two main Federal policies directly contrary to this concept were adopted, however, and were given effect either by way of expressed statutory provision or administrative interpretation. They were:

1) That the power was to be sold at the lowest possible rate which would promote its most wide-spread use, and
2) That preference in the disposition of the power was to be given to states, municipalities, rural cooperatives and public bodies, generally.

No precise standards for fixing rates were provided in any of the statutes. The statutory directions were very general and indefinite. Rates were to be fixed by the agencies so that the project would be "self-supporting and self-liquidating" or at the lowest possible rates which would promote wide-spread use. In only one statute, the Flood Control Act of 1944, were the rates to be fixed at the lowest possible amount consistent with sound business principles, and in this case the latter half of this requirement was nullified by administrative interpretation.

In some statutes an attempt was made to set up cost standards for fixing rates but in all of them important elements were omitted. Only operating and maintenance expenses, which are a very minor portion of the total cost, were provided for uniformly in those statutes where rate standards were established. In some cases, it was specified that in addition amortization of the investment be provided for, but no mention was made of interest and in

other cases the reverse was true, i.e., interest was to be provided for but amortization was not mentioned. Only in the case of the Bonneville Power Administration has an attempt been made by any agency to include in expenses the administrative and supervisory charges incurred by other governmental agencies for the benefit of these power systems.

In none of the statutes dealing with rate-making is specific provision made for Federal income taxes. Only in the case of TVA is there a separate statutory provision for the payment of a certain amount in lieu of local taxes. In TVA the payment of local taxes was designed to protect local governments from loss of revenues formerly received from tax paying private utilities whose property TVA had acquired. There was no thought of paying local taxes on property which TVA had built or would build.

In the case of the Hoover Dam project the original statute apportioned the profit from the sales of power at competitive rates to the states of Arizona and Nevada and to the Colorado River Dam Fund (a Federal fund) and when the rates were lowered by statute in 1940 these states and the fund were given certain specified "in lieu" payments of lump sum amounts to compensate them for the change in the rate formula. These "in lieu" payments can be considered as a partial offset to taxes, but they are in no way comparable with what a private utility company would pay if it owned and operated the same property.

So what we have had up to now is a very large Federal power business, involving the investment of billions of dollars, with Congress in effect acting as a Board of Directors without, however, establishing a definite policy as to the price to be charged for the products of that business or even seeing to it that the prices charged cover fully all of the costs incurred in their production.

It is always easy in any business—and popular with the customers—to sell a product for less than it is worth and less than it actually costs to produce it. The various Federal power agencies, therefore, have found it easy to adopt this low-rate philosophy.

Their concept of cost seems to be that it includes any item of expense specifically provided for by statute or involving an actual cash outlay, as for instance, payroll, or fuel payments. Under this concept, the payment of taxes not being specifically provided for by statute is not recognized either for rate making or annual accounting purposes. Similarly, if, under some statutory provision, interest is not actually paid it need not be recognized in the accounts. The TVA, for instance, pays only a relatively nominal amount of interest— $679,439 in fiscal 1954—even though its present net power investment is nearly $1 billion. This, at a 3 per cent rate, would call for annual interest payments of upwards of $30 million.

The result of all this is that Federal power is sold at less than it is fairly worth. Using fair power values in the various areas of the country where the Federal systems operate, we find that in fiscal 1953, for example, the Government sold the power it generated for $110 million to $130 million less than it was really worth. Unless present rate policies change, when presently contemplated projects are completed, this annual subsidy to consumers of Federal power could amount to $400 million a year.

Personally, I have never been able to see why the Federal Government, if it engages in any business, should sell its products for less than they are fairly worth. This is not a new idea. The Federal Government itself adopted this policy when it built its first large scale power projects—the Hoover Dam.

It should never have abandoned it. No stockholder of a private company would stand for long selling power on any other basis. In the Federal power business the Federal taxpayers are the stockholders. They have supplied all the money. They have a right to demand that this price policy be adopted.

It is often contended that the rates for Federal power should, in addition to covering other cost elements, provide as a return on investment only interest at the rate paid by the Government. I do not share this view, especially where the Government enters a field which is competitive with private industry. But even on this basis, using a long-term interest rate of 3 per cent and computing Federal, state and local taxes on the basis of payments made by non-governmental producers on production of an equivalent amount of energy, the Federal power revenues during fiscal 1953 were $75 million below the amount required to recover costs according to this standard. Over the entire history of the Federal power operations to June 30, 1953, operations failed to recover these costs by over $330 million. There can be no question that this deficiency—a subsidy to users of Federal power—had to be made up by Federal taxpayers.

Now it would be logical to assume that the Federal Government, having built these hydroelectric power plants as incidential to the main objectives of improving navigation, preventing floods, or irrigating land, would have contracted to sell power only up to capacity of the hydroelectric plants and on the basis that there would be variations in the output resulting from fluctuations in stream flow. This, however, has not happened. The TVA, for instance, requires all of its utility customers to take their entire supply from its facilities. This policy, in effect, creates an obligation on TVA to build all the generating facilities ever needed in an 80,000 square mile area which it now supplies. The result is that TVA has built large steam plants and extensive transmission systems and its distributors have no generating facilities of their own. Since TVA has, for all practical purposes, already exhausted all of its hydroelectric possibilities, the only way it can continue to meet the expanding needs of its distributors is to build more steam plants, as it has been and is doing. Much the same undertaking of a permanent utility obligation has occurred in the Pacific Northwest. In 1946 a Department of Interior brochure pointed out that the Federal Government's power activities had provided the Pacific Northwest with very low-cost hydroelectric power. It went on to say, and I quote:

> "This, in turn, has made it impractical and uneconomical, with some exceptions, for electric utilities, both public and private, within the region to develop additional electric generation in small plants. As a result, the Federal Government today has a public utility responsibility for meeting the additional power needs of the region. The program of the Federal Government has, with minor exceptions, become the single source from which all Federal power needs of the region must so far as possible be met."

I think you will agree with me that undertaking to supply power at less than cost to a large area of the country is uneconomic and can only result in a constantly expanding Federal power business with constantly increasing calls on the taxpayers for funds to pay for the necessary facilities.

The second aspect of the Federal Government's power marketing policy is concerned with the preference of one class of customers over another. A special study of the history of this preference policy was undertaken as a part of our investigation. This study showed that the preference policy had

its beginnings with the passage of the Reclamation Acts of 1902 and 1906, a period during which the conservation of natural resources was a national issue. The idea in the beginning was that the resources of the nation should be developed and utilized for the benefit of all the people. Since electric utilities were regarded at the time as monopolies with no restriction on the rates they could charge the consumer, it was concluded that the only way to insure that the benefits of developing these power resources would flow down to the ultimate consumer was to give first call to local public bodies. These public bodies were assumed not to be interested in making a profit on the transactions. Because of the absence of Federal activity in this field during the period, there were few statutes dealing with marketing from Federal power projects between 1906 and the commencement of Hoover Dam construction in 1928, but the Boulder Canyon Act covering the latter project and the legislation covering the extensive developments from 1933 to date consistently carried forward this preference policy.

This same preference policy was also incorporated in the licensing provisions of the Federal Water Power Act of 1920, which required that public bodies should have first call over private companies in the construction on navigable streams of dams for power purposes.

Not only were public bodies and cooperatives given preference to all Federally-generated power, but the statutes covering the principal projects also provided that power already contracted for by private utilities could on relatively short notice be withdrawn and transferred to these preference customers, if it were necessary to meet their requirements.

In the administration of the statutes the Department of the Interior until very recently took the position that it was under mandate, not only to contract with public bodies and cooperatives on a voluntary basis, but to seek them out and to actively promote their establishment.

As a result of this former Department of the Interior policy a large number of public utility districts took over the properties and markets of private utilities in the Columbia Basin area. In the seven-state TVA area, except for two small privately owned distribution properties, the entire retail distribution passed into the hands of municipalities and cooperatives.

Like so many government policies which are established under a certain set of circumstances, this preference policy was continued long after the circumstances had changed. It will be recalled that at the commencement of this preference policy there was no regulation of electric rates and no Federal income taxes.

Regulation of public utility rates was first undertaken in two states in 1907 and it was provided for progressively by other states thereafter. It did not become fully effective until Congress, in 1935, gave the Federal Power Commission the power to regulate interstate electric rates and the Securities and Exchange Commission the right to regulate and dissolve holding companies.

This regulation of electric utilities recognized the monopolistic character of their operations. The standards of rate making adopted by the regulatory commissions were designed to bring about the same results as would obtain were competitive forces fully operating in this field. In other words, when electric rate regulation became fully effective, as it now is, the ability of electric utilities to fix rates without restriction was ended and the original need for the preference clause disappeared.

The Federal Income Tax Law was passed in 1913 and the first tax rates were nominal, the highest rate up to the commencement of World War II

being 19 per cent, while today the corporate tax rate is 52 per cent. State and local tax rates have also advanced substantially during this period. They are about twice as high today as they were before the war.

As you know, these public bodies are themselves exempt from all Federal taxes including income taxes, and the income from their securities to the holders thereof is also exempt from Federal income taxes. The latter exemption results in lower financing costs than those met by private companies, the income from whose securities are fully taxable to a large majority of the holders. Accordingly they already have an inherent advantage over private companies.

Thus when the preference policy was first adopted there was very little difference between the tax burdens of privately owned and publicly owned utilities and it made relatively much less difference if taxes were not considered as an element of cost in fixing rates.

Today, however, the cost of Federal power can be 35 per cent to 40 per cent lower than the cost of privately sold power if corporation taxes and taxes on the income from securities necessary to finance these projects are not taken into account. For local public bodies this tax advantage is also substantially as great. So we have an entirely different tax situation than when the preference policy was initiated and its impact on power costs and rates cannot be ignored.

Now what have been the results of the enforcement of this preference policy?

Starting in 1946 when all of the principal Federal projects were in operation, the preference customers took 38.3 per cent of the power available to non-Federal users. This ratio steadily increased until in 1953 it was 52.2 per cent. Meanwhile the privately owned utilities in 1946 received 33.3 per cent of all the saleable power from these projects and in 1953, 18.5 per cent, or only about one-half the relative amount received in the earlier year. The Federal industrial customers, mostly aluminum and electro-chemical, received relatively the same amount in each year (28.4 per cent as against 29.3 per cent).

The trend toward increasing of the supply to preference users has been accelerated in recent years. For instance, in the case of the important Bonneville Administration no firm power whatsoever will be available to privately owned utilities in the Pacific Northwest after 1960 unless Congress authorizes additional projects. In 1953 privately owned utilities received 33.3 per cent of all power generated. In the case of the Central Valley project, which until recently sold 95 per cent of its output to California private utilities, from now on practically all saleable firm power will be delivered to preference customers, leaving the California private utilities with only the secondary power which such preference customers do not elect to take.

Ultimately, of course, under present practices 100 per cent of Federally generated power will go to preference customers. This preference policy in effect makes any consumer of a privately owned utility a "second-class citizen" so far as Federal power is concerned. The company from which he buys power is denied Federal power. His neighbor across the road or in the next town served by a cooperative or a municipality gets power for less than true cost and he, in his electric rate, pays local, state and Federal taxes in a greater amount because his neighbor pays nothing toward these ends.

A dramatic illustration of all this occurs in the Pacific Northwest. In 1953 power users in the state of Washington, in which there are numerous preference customers, received almost 85 per cent of the Federal power produced by the Columbia River projects and power users in the state of Oregon, where private power enterprise in electric distribution is more prevalent, received less than 15 per cent.

It is time to get back to the original intent of the preference policy and that is to eliminate the possibility of any mark-up in the price of Federal power on the resale of the same by the distributor. This can easily be accomplished through rate regulation or by contract and the policy will be much more effective than it now is.

There are many other inequities, inadequacies and faulty practices in the marketing of Federally generated power but time will not permit me to go into them. Failure on the part of the Federal Government to recognize taxes as an element of power cost and rates, coupled with Congressional direction to sell preferentially to consumers which are themselves exempt from these levies, can only result in driving out ultimately any private capital, which must assume those tax burdens--large and important as they are. There will be no net financial gain to the nation in following out the present policy because the total cost of producing the power will not be changed. The cost will merely be divided into two parts instead of one. One part will be borne by the beneficiary of the power in his rates and the other will be shifted to the general taxpayers, who in most cases will not be the beneficiary. This useless financial effort will, however, if followed to its logical conclusion, destroy private initiative, and it is not unlikely that it will also confiscate as well, indirectly perhaps, private investments already made.

I believe that thoughtful men, with the overall good of the Nation in mind, will not permit these marketing policies to continue.

Let us consider next whether all of this Federal power business is necessary to the efficient conduct of the Federal Government. Or is some or all of it a nor-essential activity competitive with private enterprise which could be eliminated?

I think there will always be debate as to whether, and as to the extent, the Federal Government should have gone into the large scale power business in the past. The past interests us only to the extent that the same valid conditions and circumstances exist today which, allegedly, may have once justified Federal entry into the power field.

It has been contended that Government activity in this business was necessary because: there was large scale unemployment; the needs of defense had to be satisfied; the Atomic Energy Commission needed large blocks of power; the hydro developments in connection with multiple-purpose projects were too large for private financing; the private utilities because of over-capitalization and faulty financial structures could not construct all the property that was needed; adequate service was not supplied to the rural areas; electric rates were too high to promote wide-spread use.

Whatever the situation may have been in the past, there is presently no unemployment problem and there has been none in the last fifteen years, there is no World War calling for emergency plant construction for defense purposes, private utilities are contributing to the extensive requirements of the Atomic Energy Commission about as much energy as the Federal agencies themselves are contributing. This is being done by entirely new plants

already built or in the process of construction—and financed entirely by private capital.

Large accumulated savings in the hands of private citizens and institutions now enable both private companies and non-Federal public bodies to obtain ample capital for the building of power plants, toll bridges, toll roads and other revenue producing property. In the thirteen years to 1953 privately owned electric companies have added $13.2 billion to their investments in properties, a sum $11 billions in excess of the total Federal investment to date. In 1953 alone these companies invested $2.6 billion, or more than the present Federal total. Local public bodies have invested to date over $2 billion in electric properties. These private and public bodies are able and willing to construct all necessary generating facilities.

The Securities and Exchange Commission has dissolved all unnecessary holding companies and sound financial structures prevail throughout the electric utility field. A study of the average rate of return earned by private utilities during recent years shows that it is practically the same as the average rate of return which commissions and courts have decided is necessary to attract capital. The rate level, therefore, has been reasonable.

If there has been any neglect of rural electric development in the past, it does not exist today. REA and the non-Federal utilities have contributed about 50-50 in installing electricity on over 90 per cent of the farms. There is no longer any necessity for REA to continue making loans in substantial and increasing amounts for enlarging facilities.

It seems clear to me, therefore, that whatever the situation may have been in the past, the private and local public electric enterprises of the country are now of sufficient financial strength, and capital accumulation in the hands of private citizens and institutions is adequate, to finance the furnishing of the country's electrical needs on the basis of earnings realized from reasonable rates. The Government's power activities are therefore now, and in the foreseeable future will be, competitive with private enterprise.

Finally, we might ask, despite this situation as it is now and appears for the future are the resources of the Federal Government so plentiful and are the burdens of Federal taxes so light that we can afford to sell Federal power at less than it costs to everyone who may want "cheap" power? I call your attention to the fact that the debt of the Federal Government at the end of fiscal 1933, when its active power program began, was only $22.5 billion; at the end of fiscal 1953 it was $266.0 billion or nearly twelve times as much. In the twenty years from 1933 to 1953 there have only been three years in which the Federal Government has not operated at a deficit.

I think the conclusion is inescapable that Federal financing of the power portion of whatever multiple-purpose construction projects may be undertaken in the future is neither desirable nor necessary. Power generation and distribution is, and essentially always has been, a local problem. Local private and publicly owned utilities presently own the thermal plants which will permit the most economical utilization of hydroelectric power. The Federal Government's deficits are still large and its debt is astronomical. There should not be any further unnecessary calls on Federal taxpayers.

The situation calls for some restraining body in the Administrative branch of the Federal Government which will curb the power construction and marketing agencies and which will decide when local interests are in fact wholly unable to undertake hydroelectric plant construction. This body, should, I think, be largely made up of non-agency or public members.

It is self-evident from the facts that non-Federal development of hydroelectric power should be actively encouraged by the Federal Government. The partnership policy, under which the Federal Government pays that portion of the construction costs which are attributable to navigation, flood control or irrigation, and local electric utilities—either public or private—pay the portion attributable to power, should be encouraged by all and emphasized and pushed by Federal legislators and administrative officials.

Proper cost and rate standards should also be covered by legislation. There should be no discrimination in favor of one class of citizens or consumers, or of one river valley or area over another. Rates for Federal power are in urgent need of revision. In order to promote economy in the public business of the Federal Government an increase in the rate level for power is a must. The accounts should be stated on the same basis as any other commercial undertaking, with full recognition of all costs, including the cost of government. The whole matter of adequate accounting and compensatory and non-discriminatory rates should, I am firmly convinced, be put in charge of the Federal Power Commission which already has such complete jurisdiction over the interstate rates of privately-owned utilities.

Most of these observations apply with equal force to present and future power projects. What should be done with present power projects? $ 2.3 billion is a lot of money to have invested in a strictly commercial business like selling power. The $10 billion total for plants in being and under way is a lot more. A necessary first step is to bring about the full earning capacity of the existing properties by increasing rates so that they will yield a fair return on the investment. It would fool-hardy to sacrifice any of the present Federal taxpayers' investment because of a low earnings level.

No hard and fast rule can be established as to the disposition of the present Federal power systems. A number of problems must be considered in connection with each one, and no one program will fit all of them. However, some plan for their ultimate disposition is clearly indicated.

I conclude with one final observation on a matter of great gravity. We are on, if not a bit over, the threshold of development of power by atomic energy. In the not-too-distant future, according to the latest estimates, a very considerable portion of our National power generation will use atomic energy as a fuel.

Whatever authority controls in one hand the power sources of an industrial nation can, if it so elects, control all industry. For this reason it seems to me urgent that our National philosophy on power be reviewed and, upon review, directed without deviation to the end that local, non-Federal organizations be the ones to develop all of our power requirements from whatever energy source—coal, oil, gas, falling water or atomic energy. In the interest of our Nation's future welfare and the perpetuation of our present form of society the Federal Government should as soon as feasible withdraw from this field as based on present energy sources and refrain from entering it based on atomic energy as a source.

JOURNAL

POWER DIVISION

Proceedings of the American Society of Civil Engineers

# WATER RESOURCES AND POWER STUDIES, TASK FORCE, HOOVER COMMISSION: RECLAMATION AND WATER SUPPLY[1]

Leslie A. Miller[2]
(Proc. Paper 899)

## FOREWORD

This paper is one of a group of five concerned with various phases of the work of the Water Resources and Power Task Force of the Hoover Commission. All were presented at the June 15, 1955, meeting of the ASCE in St Louis.

The specific function of the Hoover Commission was to study and report to Congress on ways and means of promoting economy, efficiency and improved service in the transaction of the public business in the Executive Branch of the Government. Work of the Commission was implemented by thirteen Task Forces, one of which was the Task Force on Water Resources and Power. Work of this Task Force was divided among four Task Groups assigned the following fields:

1) Power generation and distribution
2) Reclamation and water supply
3) Flood Control
4) Navigation

The five papers presented include four by the Chairmen of these Task Groups covering work of their particular groups and one by Admiral Moreell, Chairman of the Task Force. The first of these papers, "Organization and Scope; Conclusions and Recommendations," (Proc. Paper 897) by Ben Moreell, Hon, M. ASCE, explains the organization and scope of the Task Force and presents his personal conclusions and recommendations. John Jirgal in his paper, "Power Generation and Distribution," (Proc. Paper 898) cites the fact that in 1933 the Federal Government owned less than 1% of the total electric generating capacity in the United States; in 1953 the proportion was 12-1/2 % and in 1960, at the present rate, it will be 16 %. The necessity of conserving the water supply and developing an overall Federal water

Note: Discussion open until June 1, 1956. Paper 899 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, PO 1, February 1956.

1. Paper presented at St Louis meeting of American Society of Civil Engineers, Power Division, June 15, 1955.
2. Former governor, State of Wyoming, Cheyenne, Wyo.

policy is stressed in "Reclamation and Water Supply" (Proc. Paper 899) by Leslie A. Miller. In "Flood Control" (Proc. Paper 900) W. W. Horner, M. ASCE, reviews the number and cost of presently constructed or authorized, and future probable, flood-control projects. "Improvements to Navigation" (Proc. Paper 901) by Carey H. Brown, M. ASCE, is the last of the papers; it is concluded therein that many navigational improvements now under consideration or recommended are of dubious economic value.

Since the Task Force Report had not yet been released, the authors were unable to summarize the report itself or to disclose its conclusions and recommendations. They were, however, able to outline in these papers facts found in the public records and problems presented by these facts. Necessarily all opinions and conclusions are those of the individual authors.

## SYNOPSIS

Most striking features of Federal participation in reclamation and water supply are the multiplicity of agencies involved and the lack of an overall policy. Among the agencies involved in various phases of this work are the Corps of Engineers, the Bureau of Reclamation, the Department of Agriculture, the Forest Service, the National Parks, the Indian Service, the State Department and the Federal Power Commission.

The following are suggested aimed at alleviating this condition:

1) That the Congress provide a uniform policy in which all beneficiaries of all water resource developments be assessed a portion of the cost.
2) That Congress recognize as to the primary measure of the economic worth of a project, the degree to which all classes of beneficiaries are willing to pay the costs, replacing thereby the present cost-benefit analysis procedure.
3) That the Federal contribution to the economic cost (the cost plus interest) of a project should not exceed the sum of the State and local contributions. Repayment periods should not exceed 50 years after a development period, the length of which should be dependent on local conditions but not exceed 10 years.
4) That returns from power or municipal water revenues, over and above cost, should be used to subsidize all forms of water resource development, irrigation, flood control, drainage, recreation or others, but that the interest on both the investment in power or municipal water supply and the subsidized portion of the cost of other resource development, should be returned to the Treasury, together with the principal, within a 50-year period.

These suggestions could be implemented by a Board of Coordination and Review whose duties sould be as follows:

1) Be responsible for the coordination of planning water resource development by Federal agencies and by State, interstate and local agencies participating in works having partial Federal responsibility.
2) Encourage negotiation of interstate compacts under which planning for development of water resources in river basins would be initiated and directed.
3) Encourage coordination of work of Federal and State agencies in collection and dissemination of hydrologic data.

4) Evaluate adequacy of data supporting projects proposed for construction by Federal agencies and by Federal-State agencies when Federal funds are involved.
5) Evaluate the feasibility of water development proposals where Federal funds are involved and evaluate such proposals with respect to their station, regional and national implications.
6) Be responsible for application of uniform formulae for cost-benefit ratios, cost allocations, and reimbursable and non-reimbursable divisions of costs.
7) Where Federal funds are involved, provide for such field surveys and research as required to effect adequate evaluation of projects, utilizing local and State agencies where available and appropriate Federal agencies whenever required.
8) Where Federal funds are involved, recommend to Congress an allocation of participation in planning and financing of projects consistent with the respective responsibilities of local, State, interstate and Federal agencies.
9) Recommend to Congress the appropriate Federal agencies to undertake the design, construction and operation and maintenance of projects of major Federal responsibility.
10) Periodically review authorized projects and inform the President and Congress as to progress, compliance with approved plans, cost and performance.
11) Periodically review authorized projects and, where construction has not been started, recommend to the President and Congress those projects which should be deauthorized.

* * * * *

It is well known that we have in this country a limited supply of water in an average year. Various committees have from time to time pointed this up and with a steadily growing population and with increased industrial demands it is imperative that we give immediate and constructive attention to conserving the supply and using it, in all aspects, to the utmost advantage.

At the outset of its work, our Task Group on Reclamation and Water Supply was struck with the necessity to examine carefully the multiplicity of Federal agencies engaged in water development and administration in one direction and another. We were required to consider the relationships involved and the wide variations in policies, rules and regulations. We recognized, as has, for example, the Engineers Joint Council, that there does not exist any overall Federal water policy.

We have the Corps of Engineers developing water for navigation, flood control, power production and irrigation. The Bureau of Reclamation develops for irrigation, flood control, and power production. The Department of Agriculture develops for flood control, power production (REA), soil conservation, and irrigation. Added to these we have the Forest Service, the National Parks, the Indian Service, the State Department, the Federal Power Commission and other agencies involved in water development in varying phases. But over all these activities there is no guiding national policy.

With this in mind some of the members of our Group made certain findings I desire to present for consideration. We suggested:

1) That the Congress provide a uniform policy in which all beneficiaries of all water resource developments be assessed a portion of the cost. Thus, in irrigation development the farmer or water user should not be required to be the sole bearer of the burden of repayment of the cost of project. Nonagricultural areas, urban and suburban, should be required to pay an amount equitable with that paid by the farmer. In addition, the State should bear some portion of the project cost.
2) That the Congress recognize as the primary measure of the economic worth of a project, the degree to which all classes of beneficiaries are willing to pay the costs, replacing thereby the present cost-benefit analysis procedures.
3) That the Federal contribution to the economic cost (the cost including interest) of a project should not exceed the sum of the local and State contributions. Repayment periods should not exceed 50 years after a development period, the length of which should be dependent on local conditions but not exceed 10 years.
4) That returns from power or municipal water revenues, over and above cost, should be used to subsidize all forms of water resource development, irrigation, flood control, drainage, recreation, or others, but that the interest on both the investment in power or municipal water supply, and the subsidized portion of the cost of other resource developments, should be returned to the Treasury, together with the principal, within a 50-year period.

In carrying out these general recommendations, the following objectives should be adopted:

1) Reclamation by drainage and flood control, or by the recharging of ground waters, as well as by irrigation, should be provided for in any proposed new water policy.
2) A national water policy should recognize the rights and laws of the separate States concerning appropriation, use, control and development of waters within their boundaries.
3) Sovereign States have interests, rights, and obligations, which must be protected and fulfilled, so that such States and local agencies thereof should be offered the opportunity to participate in such programs and projects as are contemplated. Where interstate streams are involved, States should be encouraged and, as a prerequisite to Federal participation, be required to create interstate compacts for the purpose of dealing with water resource development.
4) Water resources should be developed in a manner not to conflict with beneficial consumptive use of water for domestic, municipal, stockwater, irrigation, mining or industrial use; and the Federal Government should assert no sovereign right with respect to one use of water over another.
5) Where the Federal Government participates in a program for the development of a major river basin, it should consider itself an agency cooperating with the States and the local agencies thereof, and that the work to be undertaken is for the benefit of such State, States, or local agencies.
6) The Federal Government should provide reimbursable loans to encourage States and local agencies as well as interstate agencies to plan, build or acquire water resource projects.

7) In order that the Federal Government may strengthen its position to assume these duties, the following provisions of Federal water policy should be established:

a) That only Congress be empowered to examine, define and authorize Federal projects.

b) Provide uniform standards by which costs, benefits, and economic justifications and related matter can be honestly submitted to the Congress, and by which subsidies, if any, may be precisely determined, and made known.

c) Avoid duplication and competition among Federal agencies engaged in water resource development.

d) Amend the power preference law to insure the administration of Federal power projects to provide fair, equitable, and nondiscriminatory treatment of all consumers and distributors of power, public and private.

e) The proceedings of All Federal and State agencies having to do with the assembly and evaluation of such basic information as topographic mapping, precipitation, stream flow, runoff, ground water percolation, pollution and all other information needed in planning such water resource projects should be properly coordinated.

f) In the field of water pollution, the Federal Government should be limited to the administration of currently existing laws, to research and to investigations, except when through inaction by a State it can be shown that water pollution adversely affects interstate waters, in which case it should be authorized to take action to alleviate pollution.

g) To avoid abuses of the past, classification of such items as navigation, flood control, fish and wildlife protection as wholly in the national interest and therefore as non-reimbursable, should be discontinued, and provision made for those benefited to absorb an equitable share of the cost of such project.

h) States, their local agencies or interstate agencies should be exempted from the licensing provision of the Federal Power Act.

i) Provisions of section 1 of the Flood Control Act of 1944, that investigations shall be conducted so as to give affected States information developed and an opportunity for consultation regarding plans and proposals and to cooperate in investigations as well as that reports must be submitted to affected States for comments, should be revised to apply to all Federal agencies authorized to plan or construct water resource projects, including the Department of Agriculture, in the prosecution of water management programs for soil and water conservation and upstream flood control.

j) The Federal Government should not undertake to supply municipal, industrial, or domestic water except where such an undertaking is an incidental part of a multiple-purpose development having to do with flood control, navigation and/or reclamation. In no case should there be a subsidy to such an undertaking. The total cost of municipal, industrial or domestic water supply should be returned to the Treasury of the United States is not to

exceed 50 years, including interest from the beginning of construction, at rates which the Federal Government is then paying on long-term borrowed money. Annual returns to the Treasury should be adequate to repay all financial costs, operation, maintenance, replacements, etc. and may in addition include sufficient funds for the partial repayment of costs properly allocable to reclamation or other features. Contracts for repayment of domestic or industrial water facilities should be made only with States or agencies of States.

k) The contribution of the Federal Government to the economic cost of a reclamation project should in no case exceed the local and State contributions. This, in most cases, is represented by the interest on the money advanced for construction of the project. The cost, exclusive of interest during construction, allocable to reclamation, should be repaid, without interest, to the Treasury of the United States within a period of 50 years after the end of a development period. Such payments should be made annually, but need not necessarily be in equal amounts. The total obligation could be paid at any time, on the basis of the present worth of the deferred payments with interest computed at the long-term loan rate currently being paid by the Federal Government.

l) All Federal investments allocable to navitation, flood control, or for fish and wildlife protection established as reimbursable should be repaid to the Treasury of the United States within 50 years of completion of the project. Such payments should be made annually but need not necessarily be in equal amounts. The total obligation could be paid at any time, on the basis of the present worth of the deferred payments with interest computed at the long-term loan rate currently being paid by the Federal Government.

m) Upon proposals acceptable to Congress, the States, local agencies, or interstate agencies should be authorized to purchase and acquire water resource projects under construction or completed. Each proposal should carry complete information concerning the project sought in order that the Congress can determine if the proposed acquisition by local agencies, a State or a group of States, is proper and in the public interest. All such acquisitions should be authorized by an act of Congress.

n) In order to facilitate (k), (l) and (m) above, upon proper authorization, loans should be made available for local agencies, States, or properly created interstate agencies.

The Task Group recognized that the present division of responsibility for water resource development among a number of governmental agencies leads to overlapping of authority with jurisdictional disputes and water resource developments in which each agency has carefully hacked out a niche for itself in the project plan.

Water resource development is a compromise of conflicting interests. Thus conflict can be expected in all phases of water resource development, and if there is to be a reasonable distribution of strength among conflicting interests, each should be associated with its own agency and pressure group. In this manner, conflicts are in the open and can be recognized.

The Task Group considers it is important, with multiple agency responsibilities, that no beneficiary of water resource development be given any undue advantage with respect to another. The adoption of the policy suggestions herein submitted would accomplish this purpose. But it is essential that the public be protected against collusion among agencies and beneficiaries. This could be done through the establishment of a board of coordination and review.

Board of Coordination and Review

The general responsibilities of the board of coordination and review would be as follows:

1) Be responsible for the coordination of planning water resource development by Federal agencies and by State, interstate and local agencies participating in works having partial Federal responsibility.
2) Encourage negotiation of interstate compacts under which planning for the development of water resources in river basins sould be initiated and directed. The appointment of Federal representatives on compact commissions should be made from among persons recommended by the Board.
3) Encourage coordination of the work of Federal and State agencies in the collection and dissemination of basic hydrologic data.
4) Evaluate the adequacy of data supporting projects proposed for construction by Federal agencies and by Federal-State agencies when Federal funds are involved.
5) Evaluate the feasibility of water development proposals where Federal funds are involved and evaluate such proposals with respect to their State, regional, and national implications.
6) Be responsible for application of uniform formulae for cost-benefit ratios, cost allocations, and reimbursable and non-reimbursable divisions of costs.
7) Where Federal funds are involved, provide for such field surveys and research as required to effect adequate evaluation of projects, utilizing local and State agencies where available and appropriate Federal agencies whenever required.
8) Where Federal funds are involved, recommend to the Congress an allocation of participation in planning and financing of projects consistent with the respective responsibilities of local, State, interstate, and Federal agencies.
9) Recommend to the Congress the appropriate Federal agencies to undertake the design, construction, and operation and maintenance of projects of major Federal responsibility.
10) Periodically review authorized projects and inform the President and the Congress as to progress, compliance with approved plans, cost and performance.
11) Periodically review authorized projects and, where construction has not been started, recommend to the President and the Congress those projects which should be deauthorized.

There was, perhaps naturally, some difference of opinion in the overall Task Force as to how a new policy would be coordinated and administered. A majority favored strengthening and expanding the present interagency Committee with a Water Resources Board which would act to perform all necessary coordinating and to establish in the Bureau of the Budget a new Board of

Review for Water Projects. The printed reports of the Task Groups and the Task Force will reveal all the detailed arguments in support of the several recommendations and these will be available in due time. What the Commission will do with the recommendations we have no means of knowing, of course, at the present time.

All the circumstances being what they are, I would be expected to give some attention to a specific project or two in the present picture of reclamation and power production. The Bureau of Reclamation, by its own promotion and without benefit of established fundamental law, has developed into an agency primarily concerned with the production of electric power rather than the irrigation of arid lands. Inasmuch as most of the really good irrigation projects available for development have been constructed or are underway, the Bureau has adopted a policy of building huge power projects the earnings of which can be utilized to subsidize the marginal irrigation possibilities which remain.

Consider a pending proposal, that of the Colorado River Storage Project. Here it is the program to build four to six large dams for power generation and to supply irrigation water to 300,000 acres of new land and supplementary water to 470,000 acres now in private ownership and cultivation. The hydropower costs would be the highest in the nation.

The average irrigation construction cost would be approximately $1,000 per acre and the Bureau of Reclamation unit reports set out that the lands are restricted by reasons of elevation, climate, soil conditions, etc. to the raising chiefly of forage crops for livestock operations. Such lands in that part of the country could not be built up, regardless of the amount of water available, to a value of more than $150 per acre in the market. The water user would be required to pay less than 15 per cent of the construction costs and power would pay the balance, without interest.

Operating on an unbalanced budget, the Federal treasury would have to borrow the money to build this project. It would have to pay 2-1/2 per cent interest on the funds. At simple compounded rates, which is the method used by the Treasury and the Bureau in figuring such matters, the taxpayers of the country would be called upon to pay interest amounting to $3,000 per acre on this project. If we should consider that the land would be worth at sale $150 per acre and place this as the direct benefit, and then for the sake of liberality grant that the indirect benefit would be an equal amount, thus fixing a value of the land in the overall general economy of $300 per acre, it will be seen that the average subsidy would amount to $2700 per acre in interest alone.

Now I would like to direct your attention briefly to another project, this in construction, the Oahe, on the Missouri River in South Dakota. This was originally conceived by the Bureau of Reclamation as an irrigation project to provide water for 750,000 acres of land in the northeastern part of that state. In the 1944 agreement with the Corps of Engineers it was taken over by the Corps in the name of flood control. The cost was estimated originally at $50,000,000. The Corps discarded the plans of the Bureau, moved the location of the dam on the river several miles and came up with an estimated cost of $ 75,000,000. The last figure I saw placed the cost at over $300,000,000.

From the beginning the feasibility of this project was challenged by numerous informed citizens of South Dakota. The protests were ignored and the dam placed under construction. So insistent were some of these people,

however, that last fall the Secretary of the Interior appointed a special panel of three engineers not in government employ to study all aspects of the project. That committee has reported but its findings have not been made public for reasons not apparent to the uninformed. I can reveal, however, that the report states 250,000 acres of the lands proposed to be irrigated are not feasible for irrigation and accordingly should be thrown out of the project. On most of the balance of the land it is recommended that drainage works be installed simultaneously with irrigation construction. Members of the panel differed in their views as to how far apart the drains would have to be placed, one contending they would have to be built in lines 200 feet apart and the others voicing beliefs that perhaps they could suffice if built 400 or 500 feet apart. In any case, you gentlemen will recognize that if these drainage works were installed along with the irrigation features, the construction costs would unquestionably be very, very greatly increased, perhaps doubled, or more.

There will be available for study in the printed report of our Task Group a tabulation giving complete information as to estimated and actual costs, estimated and actual acreages irrigated and a whole host of other figures having to do with the whole Reclamation program since its inception in 1902. I hope wide use of this will be made as it reveals some amazing information.

In conclusion, I wish to discuss two or three points raised in a recent statement circulated by the Commissioner of Reclamation and rather widely published, this taking issue with some criticisms recently advanced by the speaker and others regarding the Colorado River Storage Project.

The chief point asserted was that in directing attention to the glaring divergencies between original estimates of costs approved by the Congress at the time of authorization and final costs, the critics ignored the greatly increased costs of construction. The answer to this is in the Bureau's own records.

In 1952 the Bureau celebrated its "Golden Jubilee." In the summer of that year there was a meeting of regional employees at Cody, Wyoming. At that meeting a paper was read by a consultant to the Bureau in which these divergencies were analyzed and discussed. A careful study, according to this authority, revealed that 30 per cent of the increased costs were due to additional engineering and administration charges, 40 per cent to post-authorization changes in design and function and only 30 per cent to increased construction costs.

Another argument advanced in the Commissioner's statement is that huge subsidies for irrigation projects are justified by reason of the creation of new wealth and the additions to the rolls of new Federal taxpayers. It is the distinct reaction of the writer that if this is a valid reason for subsidies, then the argument must be carried to a logical conclusion. Any successful new enterprise creates added wealth and brings into being new taxpayers. If we are to subsidize enterprise on that score, then we must consider as consistent the subsidization of new oil refineries, new copper mines, new shoe factories.

Then there is the assertion we should build these projects because they largely pay for themselves, whereas we build flood control and navigation works which are non-imbursable. The fact is that reclamation, of itself, does not pay. A study quoted in the Task Force report cites Bureau of Reclamation figures to indicate that water users are expected to pay only

9 per cent to 30 per cent of the costs allocated to irrigation on recently proposed projects. Records of repayment disclose that it will take 300 to 400 years for some projects to repay such charges. Likewise, evidence that many flood control and navigation projects are worth building just is not to be found in Federal records. Hoover Commission Task Force on Water Resources and Power strongly contends that all beneficiaries of water development projects should shoulder consistent shares of the costs. If Congress will agree and enact the proper laws to require this, it as certain as that day follows night there will be far fewer pork-barrel authorizations thereafter and the taxpayers of the nation will be greatly relieved indeed.

But consider for a moment the implications of this argument concerning the repayment of the cost of irrigation. If a given project costs for construction $1,000 per acre and the resultant value to the general economy is $300 per acre, then it is clear someone has lost $700 per acre. It doesn't make any difference whether the cost is paid by the water user, the power purchaser or the general taxpayer, if the thing paid for has cost more than it is worth in all circumstances, then the difference between cost and value has been lost to the economy of the nation. I can draw no other logical conclusion.

Paper 900 PO 1

JOURNAL

POWER DIVISION

Proceedings of the American Society of Civil Engineers

# WATER RESOURCES AND POWER STUDIES, TASK FORCE, HOOVER COMMISSION: FLOOD CONTROL[1]

W. W. Horner,[2] M. ASCE

## FOREWORD

This paper is one of a group of five concerned with various phases of the work of the Water Resources and Power Task Force of the Hoover Commission. All were presented at the June 15, 1955, meeting of the ASCE in St. Louis.

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1. Paper presented at St. Louis meeting of American Society of Civil Engineers, Power Division, June 15, 1955.
2. Cons. Eng., Horner & Shifrin, St. Louis, Mo.

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## SYNOPSIS

In summary, the authorized flood control program has involved 1033 projects, with an estimated cost of over $11 billion. Looking beyond the authorized program the Corps of Engineers in 1948 made a forecast of ultimate flood control which might cost as much as $25 billion. In addition the Department of Agriculture is contemplating projects which could involve an ultimate capital expense of $17 billion.

Report of the Chief of Engineers for 1951 showed that the estimated Federal cost of Corps flood control projects which would be substantially completed by July 1, 1952 amounted to a little more than $3 billion and local interests were expected to contribute only about 5.8 per cent of that amount. Data furnished the Task Force showed that local direct beneficiaries pay an even smaller percentage of the construction costs of headwater flood dams built by the Department of Agriculture.

Normally Congress authorizes flood control projects on receipt of a report from the administrative agency showing that benefits to accrue appreciably exceed the cost. In some instances, however, Congress has authorized projects without requesting a report of costs and benefits. Further, on occasions the project constructed is at wide variance with the one authorized by Congress.

The 1936 Flood Control Act, upon which Federal flood control construction is based, set up a policy that all flood control projects should add to the National Wealth; that is benefits created should exceed the wealth diverted as construction cost, but Congress failed to make any specific provision for determination of either cost or benefits. Thus the burden of determination of both cost and benefit falls on the construction agencies who have been forced to develop their own procedures. As a result there is wide variance between different agencies as to benefits accruing to the same flood plain and generally stated benefits seem unrealistically high. It has been found on occasions that benefits were claimed for a project after these same benefits had been fully credited to other projects.

* * * * *

Floods take a tremendous annual toll in lives and property destruction, and the Nation has poured vast sums of money into measures to prevent their recurrence or lessen their harmful effects.

But has this money been spent wisely? Have we got the best flood control for the least money? The Task Group on Flood Control has examined the

flood control activities of the Federal Government from the standpoints of policy, administration, and organization in order to provide answers to these questions.

Since the Commission has not yet released the Task Force report, it is not possible at this time to present and discuss its conclusions. I can, however, discuss the procedures employed by the Task Group, some of the factual data which it accumulated, and present some personal comments with respect to it.

Members of this Task Group, in addition to participating in the meetings and hearings of the Task Force, as a whole, had separate conferences with the Chief of Engineers and his staff and with the Assistant Secretary of Agriculture. It also met with division and district engineers in several parts of the country, received presentations from them, and visited work under construction. It also had an extensive meeting and inspection trip with engineers and officials of the Soil Conservation Service in Texas where the upstream flood control program is well under way. It held numerous conferences with state officials having responsibility in the water field. Both the Corps of Engineers and the Department of Agriculture officials made formal presentation of their programs and procedures before the Task Force as a whole. In addition to the material voluntarily presented by these agencies, the Task Group requested additional data and supplementary statements. I would particularly like to compliment the Office of the Chief of Engineers on its compliance with requests for additional specific information, much of this involving tedious compilations from records in division and district offices.

The Task Group was given an adequate and expert staff, and in addition, was permitted to call in consultants on particular matters, some of which involved detailed examination of procedures in the field.

Before discussing the flood control program in detail, it might be well to have some perspective on the program size. The Office of the Chief of Engineers prepared compilations of the projects authorized by Congress, beginning with the Flood Control Act of 1936, but including also the Lower Mississippi Project. In summary, the authorized flood control program has involved 1,033 projects, with an estimated cost of over 11 billion dollars. At the beginning of this year, four hundred and twenty-three had been completed or were under construction, with an up-to-date cost estimate of five and three-fourths billion. This leaves a backlog of six hundred projects, now estimated to cost about five and one-half billion, which have been authorized, but not started. The Chief of Engineers has now classified 390 of these projects as active and ready for construction with a round estimate of about three and one-half billion, and 310 projects estimated at over one billion as inactive or deferred for further study. It would appear that if Congress continues appropriations at about the rate which has recently been current, it will take something like twelve years to complete the active program for which Congress has made the Corps of Engineers responsible.

Looking beyond the presently authorized program, the Corps of Engineers in 1948 made a forecast of ultimate flood control which Senator Malone introduced in the Congressional Record. When the estimated ultimate cost of flood protection as set out therein has been transformed to current prices, and a proportionate allowance made for other features which would be involved in the multipurpose projects of such a program, we arrive at a possible capital cost of 25 billion, or about two and one-half times the program as presently authorized.

The 1936 Act provided that certain upstream work would be assigned to the Department of Agriculture. Under this provision, Congress has authorized projects estimated at $175,000,000. In 1954, Congress passed the Watershed Protection and Flood Prevention Act, which took the Department of Agriculture out from under the provisions of the Flood Control Acts, and instead give it independent responsibility to proceed with an upstream program. On the basis of need for upstream watershed work as foreseen by the National Association of Soil Conservation Districts, and using current costs, it has been estimated by the staff that this program could involve an ultimate capital expenditure of about seventeen billion dollars.

It should be further noted that neither the prospective program under the Corps of Engineers, nor the seventeen billion under the Department of Agriculture represent the real economic cost. These are only the initial capital outlay. If these are transformed into values covering interest and amortization, the total sum approaches sixty-five billion dollars, or the equivalent of nearly one fourth the national debt.

I have no idea that any such program ever will be carried out. Undoubtedly, many of the units could not be economically justified, but even if heavily discounted in this way, there is still a potential expenditure on the order of fifty billion dollars.

Who Pays the Bill?

The report of the Chief of Engineers for 1951 showed that the estimated Federal cost of Corps flood control projects which would be substantially completed by July 1, 1952 amounted to a little more than three billion dollars, and local interests were expected to contribute only about 5.8 per cent of that amount. Data furnished the Task Force showed clearly that local direct beneficiaries pay an even smaller percentage of the construction costs of headwater flood dams built by the Department of Agriculture.

To summarize at this point, it is clear that we have a flood control program involving tremendous cost, paid for almost entirely by the Federal taxpayers, with the actual beneficiaries making no more than a token contribution even though at the present time, the beneficiaries would be financially capable of paying substantial parts of the cost. In view of this, the Task Group found it desirable to investigate the background from which this situation has developed, and made a detailed review to that end.

The 1936 Act stated for the first time that the Federal Government assumed the responsibility for the abatement of flood damages anywhere in the United States; that the Federal taxpayers would pay the cost, except for a nominal percentage covering rights-of-way, easements, and in some cases, operation; and that projects would be undertaken whenever the benefits to whomever they accrued exceeded the cost.

The record shows that the Act was passed under what appears to be almost hysterical conditions related to devastating damages in certain major cities out of the floods of the middle thirties, to the recognition of the depression situation, when most of the cities were on the verge of insolvency, and the need for a work relief program. None of these factors are operative today. Yet the policies which they brought into being are <u>still in effect.</u> This is a striking illustration of the fact - often stated by students of bureaucracy - that government programs and policies, once initiated, rarely recede from their high-water mark; even when the causes which gave rise to them have disappeared.

The act was strongly opposed by some of the more conservative members of Congress, and the then Chief of Engineers, General Markham, insisted that the Federal Government should not pay the entire cost. The late Senator Vandenburg stated in the course of the debate:

> "Let it not be overlooked that this is the first time in 150 years of American history when it has been proposed to assert that floods upon practically all the rivers of the United States constitute a menace to national welfare and are a Federal responsibility **********. The human imagination can hardly encompass the total extent of the burden and responsibility which is thus laid at the door of the Treasury of the United States by the adoption of this policy."

Agencies of the Federal Government plan, build and sometimes even contribute to the maintenance and operation of minor works of purely local or intrastate utility. To cite some specific examples: Congress authorized the Bureau of Reclamation to construct the Sanford Reservoir on the Canadian River to provide a source of municipal water supply for the City of Amarillo, Texas, and nearby communities. Congress authorized the Corps of Engineers to provide relief from a "serious basement flooding" problem confronting ten communities adjacent to Detroit in Oakland County, Michigan. And the Soil Conservation Service has, in many parts of the country, constructed headwater reservoirs, at Federal expense, to protect from flood damage, bottomland areas of such limited extent that the bulk of the benefits accrue to a very few farmers, or sometimes a single owner.

To summarize again at this point, it would seem that after twenty years of experience with the 1936 Act and its amendments, the time has come when the Congress, with reasonable consideration of the Federal taxpayers, should take a new and objective look at the whole question of Federal flood control responsibility. In my opinion, the national policy should be restated to restrict the program to major projects of national significance, and to require the direct beneficiary to agree to pay a substantial part of the cost.

## Legislation and Administration

The record shows that in general Congress authorizes flood control projects on receipt of a report from the administrative agency showing that the benefits to accrue appreciably exceed the cost. However, Congress itself is guilty of spectacular violation of this procedure, in certain instances, having authorized projects without even requesting a report on costs and benefits. This is true, for example, of the Los Angeles flood control program, and of some thirty large reservoirs which Congress authorized by the Flood Control Act of 1938, without waiting for the Corps of Engineers to determine their economic justification.

The authorizing legislation is also very broad in character, generally covering a project which has been reported upon, but "with such modifications as the Chief of Engineers may find desirable."

This language has thrown on the agencies an unreasonable requirement for administrative decision. For example, the Chief of Engineers may find and has found it desirable, instead of one authorized reservoir, to substitute three at different locations, at different costs, and possibly with appreciably different benefits. Under such circumstances, in some instances, the agencies have presented a new report to Congress; but under other conditions,

have decided to proceed under the original authority. It would certainly appear that legislation should be more definite and final.

Upon occasion the difference between what the Congress authorized and what the Federal agency actually did was so great that it seemed to constitute a deliberate disregard of the law. A striking example is afforded by the operations of the Department of Agriculture under the Flood Control Acts. The 1944 Act authorized the Department to carry out, in eleven river basins, programs made up of soil conservation, forestry and other land-treatment measures. The funds appropriated under this legislative authority were, however, largely used to construct headwater reservoirs and other engineering works not included in the authorized programs. Several years later Congress was persuaded to legalize this substitution of an engineering for an agricultural program, but the fact remains that in the interim the law had been almost completely disregarded.

Economic Justification

The 1936 Act set up the policy that all flood control projects should add to the national wealth; that is, the benefits created should exceed the wealth diverted as construction cost, but the Congress failed to make any specific provision for the determination of either cost or benefits. This, in effect, placed the burden of such a determination on the construction agencies. These agencies would normally be expected to be expert in estimating costs, but would not normally be qualified to appraise benefits. They have, however, been forced to develop their own procedures in the latter matter.

Estimate of Cost

Committees of Congress, from time to time, have complained that costs have been underestimated, particularly by the Corps of Engineers. The Appropriations Committee of the House in 1951 found that for 182 projects, the cost estimated at the time of authorization had increased by over three billion dollars, to about two and one-fourth times the original estimate. The Corps of Engineers has pointed out that the greater part of this increase could be attributed to inflation. The Task Group restudied this matter, and agreed to this conclusion, but still found that the cost had otherwise been increased about twenty-five per cent, enough, for many projects to upset the original finding of economic justification.

I am glad to be able to report that this matter related to projects in the early part of the program, and that in recent years, authorization estimates appear to have been liberal.

Method of Appraisal of Benefits

The Task Group studies showed a quite unsatisfactory situation, particularly where appraised benefits rested on estimated damages to the agricultural lands; here the methods employed appeared quite generally to produce stated benefits that seemed unrealistically high. It was reported to the Task Group, that for the same flood plain, such benefits had been separately appraised by the Corps of Engineers and the Soil Conservation Service, with variations between the two of as much as one hundred per cent in either direction, but on the average, the estimates of Soil Conservation Service were on the order of four times as high as those of the Corps.

One expert reached the conclusion that the estimates of both agencies were generally inflated to such a degree that economic valuations had become virtually meaningless.

The Task Group also found instances of apparent blind spots where the agency applying its own formula continued to apply it after benefits already had been fully credited to other projects. An examination of one group of twenty-six reservoirs indicated that twenty of these could not have been reported as economically justified if such benefits had not been applied; but they were so reported and were authorized by Congress.

From such instances, it might be indicated that Congress should provide for an independent appraisal of benefits by particularly qualified groups, and might well also have an independent review of the findings prior to authorization.

Factual data available to the Group indicated several other cases of what appeared to be unsatisfactory procedure. These covered such matters as failure to consider lower cost alternative solutions of flood problems, unsatisfactory scheduling of project construction, and in some instances, overexpensive project design.

Another matter that would seem to justify considerable concern is the development within the Department of Agriculture of a large new engineering organization to handle the design and construction of so-called upstream reservoirs, many of which are of considerable size. There had been much controversy in the past with regard to the overlapping and inefficiency in the work of the Bureau of Reclamation and the Corps of Engineers, both handling large dams. There would seem to be a serious question whether Congress should provide for three large engineering organizations in the same general field.

Time does not permit mention of many other matters that came before the Group, and I regret very much that we are not permitted to discuss this subject in terms of conclusions and recommendations. I am hoping, however, that the factual data and the comments I have been able to offer will give some background for a further study of the report when it is available.

JOURNAL

POWER DIVISION

Proceedings of the American Society of Civil Engineers

# WATER RESOURCES AND POWER STUDIES, TASK FORCE, HOOVER COMMISSION: IMPROVEMENTS TO NAVIGATION[1]

Carey H. Brown,[2] M. ASCE
(Proc. Paper 901)

## FOREWORD

This paper is one of a group of five concerned with various phases of the work of the Water Resources and Power Task Force of the Hoover Commission. All were presented at the June 15, 1955, meeting of the ASCE in St. Louis.

The specific function of the Hoover Commission was to study and report to Congress on ways and means of promoting economy, efficiency and improved service in the transaction of the public business in the Executive Branch of the Government. Work of the Commission was implemented by thirteen Task Forces, one of which was the Task Force on Water Resources and Power. Work of this Task Force was divided among four Task Groups assigned the following fields:

1) Power generation and distribution
2) Reclamation and water supply
3) Flood Control
4) Navigation

The five papers presented include four by the Chairmen of these Task Groups covering work of their particular groups and one by Admiral Moreell, Chairman of the Task Force. The first of these papers, "Organization and Scope; Conclusions and Recommendations," (Proc. Paper 897) by Ben Moreell, Hon, M. ASCE, explains the organization and scope of the Task Force and presents his personal conclusions and recommendations. John Jirgal in his paper, "Power Generation and Distribution," (Proc. Paper 898) cites the fact that in 1933 the Federal Government owned less than 1% of the total electric generating capacity in the United States; in 1953 the proportion was 12 1/2% and in 1960, at the present rate, it will be 16%. The necessity of conserving the water supply and developing an overall Federal water

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1. Paper presented at St. Louis meeting of the American Society of Civil Engineers, Power Division, June 15, 1955.
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policy is stressed in "Reclamation and Water Supply" (Proc. Paper 899) by Leslie A. Miller. In "Flood Control" (Proc. Paper 900) W. W. Horner, M. ASCE, reviews the number and cost of presently constructed or authorized, and future probable, flood-control projects. "Improvements to Navigation" (Proc. Paper 901) by Carey H. Brown, M. ASCE, is the last of the papers; it is concluded therein that many navigational improvements now under consideration or recommended are of dubious economic value.

Since the Task Force Report had not yet been released the authors were unable to summarize the report itself or to disclose its conclusions and recommendations. They were, however, able to outline in these papers facts found in the public records and problems presented by these facts. Necessarily all opinions and conclusions are those of the individual authors.

## SYNOPSIS

Many navigational improvements currently recommended are "fringe" projects of dubious economic worth, having little general or even regional worth. Money is being spent on local projects backed by promotional organizations thus diverting money from more worthy projects.

Two basic issues: Who is to pay for the improved waterways which are provided at public expense for the benefit of shippers and carriers who use them; and what type of service on the part of these users should the public rightfully expect?

At present time almost all river and harbor improvements are provided and operated exclusively by the Federal Government. In cases where such projects are of local significance it seems that financial responsibility for construction and maintenance should rest with the localities and the individuals receiving benefits. This has not been the case; local contributions, except for grants of rights-of-way, spoil disposal areas, adjustments of local highway and drainage facilities, and the like, have amounted to only about 2 per cent of the total cost of river and harbor projects.

To date Federal Government has spent $4.5 billion to improve, operate and maintain its navigation system and current estimates of future cost of authorized projects exceed an additional $4.5 billion. Annual maintenance cost of existing system is $85 million.

Inland waterways which have proved of outstanding economic value to the country, are those which carry large quantities of coal, iron ore, iron and steel products, petroleum and grain. Harbor developments in ports which have extensive rail, highway, waterway and pipeline connections with large productive regions are worthy of Federal support.

Where either inland or harbor developments benefit only a small local area or where they are (as happens at times) planned with the object of securing lower railroad freight rates no Federal expenditure is justified.

Another regretable tendency is allocation to navigation of a disporportionate share of the cost of multiple purpose projects.

* * * * *

Whenever Congress, beset with the difficult problem of providing enough money for national defense and for other essential functions of government, finds funds it can divert to water conservation measures, it is still confronted with the necessity of selecting a limited number from a long list

recommended by the executive agencies responsible for flood control, reclamation, power development, and improvements to navigation.

Current procedures governing those recommendations frequently overlook the paramount public interest and lead to economic waste. A substantial number of navigational improvements currently recommended are obviously "fringe" projects of dubious economic value, having little general or even regional worth.

If the water conservation and development program is to go forward, should attempts be continued to distribute projects among as many Congressional districts as possible? Should scarce funds be devoted to worthy coastal harbors, the Gulf Intracoastal Canal, the Mississippi-Ohio inland waterway system, and other waters of recognized economic value, or should they go to more of less obscure local projects of little general significance, because of strong cumulative pressures of promotional organizations?

All of us recognize the wisdom of developing and protecting the truly indispensable water resources of this country, but we resent the expenditure of our money on worthless enterprises. Too much is being spent on projects of minor utility; too much on projects inspired by local self-interest; too much in particular on navigation projects that lack both economic justification and national significance; too much to maintain obsolete and useless works; and too much to make navigable at great costs streams which in their natural state are not and never have been navigable, and which are not likely to make a substantial contribution to transportation.

In the planning, financing, construction, and operation of the inland navigation program, the cardinal principles and stated requirements of the national transportation policy, as expressed by Congress in the Transportation Act of 1940, have been flouted to the advantage of the water carriers and shippers, generally to the disadvantage of other forms of transportation, and probably also to the detriment of other shippers.

The present practice of bestowing subsidized facilities carries such appealing advantages to favored beneficiaries that other potential recipients are encouraged to press for extensions and enlargements of the existing system with little regard for the general economic effect.

The basic issues are two: Who is to pay for the improved waterways which are provided at public expense for the benefit of shippers and carriers who use them; and what type of service on the part of these users should the public rightfully expect?

The party with the dominant interest is the taxpaying public. The facts, the contentions, and the justice of the case must be examined on this indisputable premise. Is it reasonable to expect the American public to put out vast sums each year to provide free improved inland waterways to be used by a limited number of shippers and carriers for their own profit without any reimbursement to the Federal Government?

Under this present policy, almost all river and harbor improvements are provided and operated exclusively at the expense of the Federal Government. Many navigation projects now in use and many more that are being considered for authorization or appropriation, are largely, if not altogether, local in their utility. A large share of financial responsibility, not only for the construction, but also for maintenance and operation of these projects, should obviously rest with the localities and the individuals receiving the benefits, rather than on the general taxpayers, most of whom realize no advantage from the improvements. But the contribution from State and local sources, except for grants of rights-of-ways, spoil disposal areas, adjustment of local

highway and drainage facilities, and the like, has amounted to only about two percent of the total cost of river and harbor projects.

This policy has led to the authorization of many projects of questionable economic merit, which in all probability would not have been undertaken had the local promoters been required to back their exaggerated claims with cash. The preponderantly local significance of many navigation projects recommended to Congress by the Corps of Engineers, is aptly illustrated by the Rivers and Harbors Bill introduced in the House of Representatives on May 12, 1954.

The most pervasive of the causes of unhealthy decline in the vitality of State and local governments is the crystallization of the idea that the Federal Government in Washington should provide the money to satisfy any and every local need and desire. It is evident that local concern as to the economic soundness of any project or program diminishes in proportion to the decrease of local responsibility for its financing. Rather unaccountably this is true even in those areas where contributions to Federal revenues exceed the receipts therefrom. The water resources conservation and development program is a conspicuous example of this condition and therefore presents an opportunity to reverse a bad principle and a dangerous practice.

The Federal Government has already spent more than $4.5 billion to improve, operate and maintain its navigation system, including 286 coastal harbors, 131 Great Lakes harbors and channels, the intracoastal canals, and numerous rivers. The navigable waterways of the United States, as the term "navigable" is used in legal parlance, are estimated to be 60,000 miles in length, of which 28,400 miles have been authorized for improvement, and 22,500 miles have actually been improved for navigation.

The current estimate of the future cost of completing authorized navigation projects exceeds an additional $4.5 billion. More billions will undoubtedly be required within the next few decades to take care of extensions,enlargements, and improvements. The annual maintenance costs of the existing system now exceed $85 million and may be expected to grow steadily as the system is extended.

The more important coastal and Great Lakes ports and channels, the Gulf coastal canal, and the Mississippi, Ohio, Monongahela, and Illinois River systems provide water transportation for a vast amount of commerce, and, with a limited number of other waterways, have proved to have outstanding economic value for the country. The inland waterways that have given an impressive demonstration of economic utility are those which carry large quantities of coal, iron ore, iron and steel products, petroleum, and grain. Streams which do not develop a large volume of traffic in those commodities do not as a rule sustain a favorable benefit-cost ratio for navigation improvements.

Harbor developments also vary widely in their economic soundness and importance to the national welfare. Ports which have extensive and efficient rail, highway, waterway, and pipeline connections with large productive regions evidently merit a large degree of Federal support. But many harbor projects serve restricted territories, and in many cases the tributary areas overlap extensively and compete for the same maritime traffic. Many small fishing and recreational harbors improved at Federal expense are purely local in character and serve no useful national or regional purpose.

All but a small percentage of the carriers on inland streams are private and contract carriers, relatively few of which are subject to regulation by

the Interstate Commerce Commission. No inland water carrier assumes the obligation of a common carrier, as that term is applied to railroads and truck lines. The general public cannot get its goods shipped on these inland waters, and only a limited number of shippers, mainly industries situated on or near these improved waters, enjoy the benefits of Federal expenditures for such improvements. It is altogether possible that the carriers on inland waters could not maintain their present low charges for transportation if they were required to serve the public as common carriers in the same comprehensive manner as are the railroads. The public's money is being spent to provide navigation channels for carriers which do not recognize an obligation to serve the general public.

A substantial number of projects are being operated without real advantage to the commerce of the country, and with the Federal Government absorbing the cost. No satisfactory inventory of all such uneconomic projects, with the history of their performance, is available, but many of them have been identified in various reports of the Corps of Engineers. There is no effective machinery for closing down navigation projects which are admittedly not worth maintaining. Until recently, no provision had been made for full or regular audits of the navigation projects maintained at Federal expense. No private business could safely operate without an accurate system of accounting.

As far back as 1927, the Corps of Engineers listed 131 projects which had either outlived their usefulness or had failed to develop a volume of traffic sufficient to justify their continued maintenance, and recommended their abandonment. To date Contress had taken no action on this recommendation.

Federal navigation projects have brought added expense to State and local governments by reason of bridge clearances and other conditions which their construction imposes.

One reason navigation interests are able to get so much money for undeserving projects is that public power advocates and those favoring flood control seek to charge off to navigation substantial portions of the costs of such projects.

The Corps of Engineers has relied to a greater or lesser extent on "indirect", "collateral", and "social" benefits to support recommendations for navigation projects. It has also relied on these undefined and indefinable yardsticks to justify recommendations for the continuation of navigation projects of little tangible value.

Adequate user charges would go a long way to correct most of these unsatisfactory conditions. With user charges, navigation expenditures would be investments rather than expenses; discrimination between different forms of transportation would be mitigated if not entirely eliminated; and uneconomic projects would stand little chance of approval. The charges should be adequate to reimburse the Federal Government at least for the cost of maintaining and operating existing improvements. It is also advocated that charges sufficient to retire the capital costs of new undertakings in the field of navigation improvement be borne by those directly benefited.

Navigation projects are at times promoted by local interests with the avowed object of securing lower railroad freight rates. The financing of unsound navigation projects to promote hydroelectric, irrigation or flood control projects is also wasteful. It is axiomatic that no navigation project be undertaken unless it can be justified for that purpose. The opportunity to allocate to navigation a portion of the costs of a multiple-purpose project

holds out to those interested in power a temptation to charge too large a portion of the costs to navigation and thereby lay the foundation for cheaper power. The allocation of 50 per cent of the costs on the Columbia River between Portland and the Dalles is such an instance. In other instances, excessive allocations have been made to flood control, as in the case of the Southwestern Power Administration, to the end that Power might be sold at less than cost.

Reclamation, flood control, and power projects, single or in combination, should stand or fall on their own merits, and should not be propped up by an economically feeble navigation feature merely because it provides a "free money ride".

The over-zealous quest of public money for works of mainly local utility and doubtful merit is the greatest single obstacle to a sound water resource development program. Apparently no executive agency has been able to resist these raids on the Treasury. A program consistent with the best interests of all the people cannot be sustained until projects are subjected to analysis by an independent agency endowed with broad, national vision and inspired by a determination to place the general welfare above the claims of subsidy-seeking groups, and above the ambitions of any Federal agency.

The second obstacle is the natural competition among Federal agencies for funds, prestige, and power. These agencies have from time to time recommended unsound projects. These projects have been appraised by competing agencies whose judgment could well be swayed by a desire to enjoy the distinction of building and operating the projects. There is an obvious need to examine and to evaluate navigational projects in their relation to the whole economy and particularly from the point of view of the transportation system of the country.

It is interesting to note that the search for a brief and effective statement of principle to be recommended as a guide for administration and development of the navigation system disclosed such a statement in the declaration of policy incorporated by Congress in the Transportation Act of 1940. It reads as follows:

> "It is hereby declared to be the national transportation policy of the Congress to provide for fair and impartial regulation of all modes of transportation,* * *, so administered as to recognize and preserve the inherent advantages of each; to promote safe, adequate, economical and efficient service and foster sound economic conditions in transportation and among the several carriers; to encourage the establishment and maintenance of reasonable charges for transportation services without unjust discriminations, undue preferences or advantages, or unfair or destructive competitive practices; to cooperate with the several State and the duly authorized officials thereof; and to encourage fair wages and equitable working conditions—all to the end of developing, coordinating, and preserving a national transportation system by water, highway, and rail, as well as other means, adequate to meet the needs of the commerce of the United States, the Postal Service, and the National defense."

Application of these sound principles to our navigation system is unfortunately lacking in many respects. Steps to insure such application are urgently needed.

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Discussion of
"MODEL TESTS, ANALYTICAL COMPUTATION AND OBSERVATION OF AN ARCH DAM"

by M. Rocha, J. Laginha Serafim, A. F. da Silveira, and J. M. Ressurreicao Neto
(Proc. Paper 696)

MERLIN D. COPEN.[1]—This paper reports one of the very few available comparisons of model and prototype studies of deformations and stresses in a concrete dam. The authors should be complimented for their lengthy study and extensive research in obtaining the data and preparing the material reported in this paper.

In view of the obviously careful and detailed efforts expended in obtaining the reported material from the model and prototype, the question arises as to why these results were then compared with an admittedly incomplete and approximate trial-load stress analysis; "In the trial-load analysis only the radial displacements were considered * * *." The effects of tangential shear and twist cannot be disregarded if an unbiased comparison of model and analytical methods is to be made. Figure 1 illustrates the radial displacement of an arch element in the proposed Yellowtail Dam considering only radial deflection agreement as compared with the radial displacement including the effects of tangential shear and twist. Table 1 is a comparison of arch stresses at the crowns and abutments of horizontal elements in this same dam showing the effects of tangential shear and twist. The effects of tangential shear and twist on stresses in the crown cantilever are illustrated in Table II. Since principal stresses are not normally computed on the basis of radial displacements only, the effects on them of tangential shear and twist are not available.

The authors further state that, "A first approach having led to fairly high tensile stresses at the base of the cantilevers, a more accurate analysis was carried out on the assumption that the cantilevers cracked at the base." It would be interesting to know how the extent of cracking was determined since this requires a rather definite knowledge of the tensile stress necessary to produce failure in the concrete. Furthermore, as shown in Table II, the effects of tangential shear and twist often show a reduction in tensile stress such that an assumption of cracking would not be necessary.

The authors state: "In the United States the good performance of Carlson Meters, together with a better knowledge of the properties of concrete, enabled further development by the Bureau of Reclamation, so the time evolution of the stresses at nine points in the base of Shasta Dam was computed from readings. Reliable stress measurements were also obtained by Tennessee Valley Authority at Norris, Hiwassee and Fontana Dams by Carlson Meters. It should be noted, however, that in all those investigations no attempt was made to analyze the stresses by computation or model testing." The difficulties encountered by foreign engineers in obtaining all

1. Engr., U. S. Bureau of Reclamation, Denver, Colo.

available published material in the United States is realized. It is somewhat surprising, however, that at least in the case of Shasta Dam the authors were not acquainted with the extensive analytical stress studies which have been made. Reference is made here to only one study, but several design studies, based on both the linear and nonlinear assumption, have been made.(1) The results of stress computation on Norris Dam(2) by the Bureau of Reclamation, and stress analysis by the TVA of Hiwassee Dam(3) may also be obtained.

The last two paragraphs in the paper say: "Seeing the high standard achieved in model testing, in accuracy, in rapidity and in economy, it is stated that, except for minor projects, model studies should be carried out. The trial-load analysis can be used, in principle, with any shape and conditions of foundation. However, the necessity for reducing costs and time requires simplifications in the number of arches and cantilevers and adjustments, which leads to results of unknown accuracy, especially with dams exhibiting dissymmetry, irregular foundation lines, heterogeneous foundations, shape singularities such as cracks, bulky abutments, spillway openings and the like.

The only way of securing dependable information on the practical use of the analytical methods will be by systematically comparing their results with observations of models and prototypes."

In view of the fact that the specific time and expenditure of funds required for the model tests are not reported in this paper, it is difficult to understand the first part of this conclusion. The paper does say: "The investigations were started 1947, their experimental part being completed 1951." The paper later states: "A later detailed report will cover the whole series of model tests carried out at the laboratory during the last 8 years." These statements indicate a considerable lapse of time and expenditure of funds. This makes the criticism of the trial-load method on the basis of time and cost seem a little unnecessary. The Bureau of Reclamation with its available supply of experience, knowledge and trained personnel has not found the time and expense of the trial-load analysis excessively high. On the contrary the savings in materials made possible by use of trial-load analysis in design studies has made the cost of the analysis become insignificant. By use of experience gained from numerous previous stress studies it is possible to base designs of dams on greatly simplified trial-load studies, such as a simple adjustment of radial displacements of the crowns of arches and a crown cantilever. When a design containing the most desirable features, both from a structural and economical standpoint, has been determined, then a detailed trial-load study is made and any changes found necessary from its results are incorporated in the final design of the dam.

It should be stressed that the greatest value derived from the trial-load method of analysis is not necessarily the final numerical values of displacements and stresses obtained, but rather the determination by comparative studies of the most economical structure under a given set of conditions. Where the necessary design data are available the actual time required to make a complete trial-load study, including tangential shear and twist, varies from 3 to 6 months depending on the conditions encountered. Generally 3 men, trained and thoroughly familiar with trial-load, are required for the work involved in the analysis.

With the use of modern construction methods and grouting procedures the objection to "irregular foundation lines" and "heterogeneous foundations"

may largely be eliminated by careful excavation and preparation of the abutment and foundation area. But even if these conditions are encountered it is doubtful if the accuracy obtainable from small-scale models is greater than that possible by use of trial-load procedures. In studies completed in the past a large variety of foundation conditions have been met and successfully included in the analysis with very little additional time required. It is, in fact, safe to say that any or all of the difficulties mentioned as adding too much time and cost to trial-load studies can and have been included in analyses with little additional effort involved. "Shape singularities such as cracks" may be easily avoided by careful design. In this regard the use of trial-load has some very definite advantages in that through the use of simplified methods and the application of experience and judgment, areas of tension may be reduced or eliminated without a detailed study of any kind.

It might also be stated in defense of the trial-load method that the application of dynamic loads, such as earthquake, and temperature variations is easily included, while such loads are extremely difficult or impossible to handle with any degree of accuracy on models. The authors were fortunate in having an ideal condition where these loads did not create a problem.

The final paragraph is extremely perplexing since the authors are acquainted with the "remarkable investigations carried out on plaster and rubber models of Hoover Dam."[4] These investigations proved rather convincingly the practical value of trial-load and from these results it would not appear that such comparisons would have any further value except for unusual and difficult problems.

Von K. Goriupp in an article on Hierzmann Arch Dam[5] says: (as translated) "As mentioned previously, only the analysis for the radial deflections were made. These results when compared with those had by experiment can be considered good in agreement. The deflection and stress diagrams show in both cases the same tendency and their numerical values give for comparisons such as these very satisfactory correspondence."

In view of the very brief evidence presented in this discussion, the exhaustive model tests on Hoover Dam, and actual measurements of dams in service by the Bureau of Reclamation, the conclusions arrived at in this paper regarding the trial-load method of analysis seem to be unjustified. Perhaps a more complete study with the aid of more experience in the use of trial-load would be of value.

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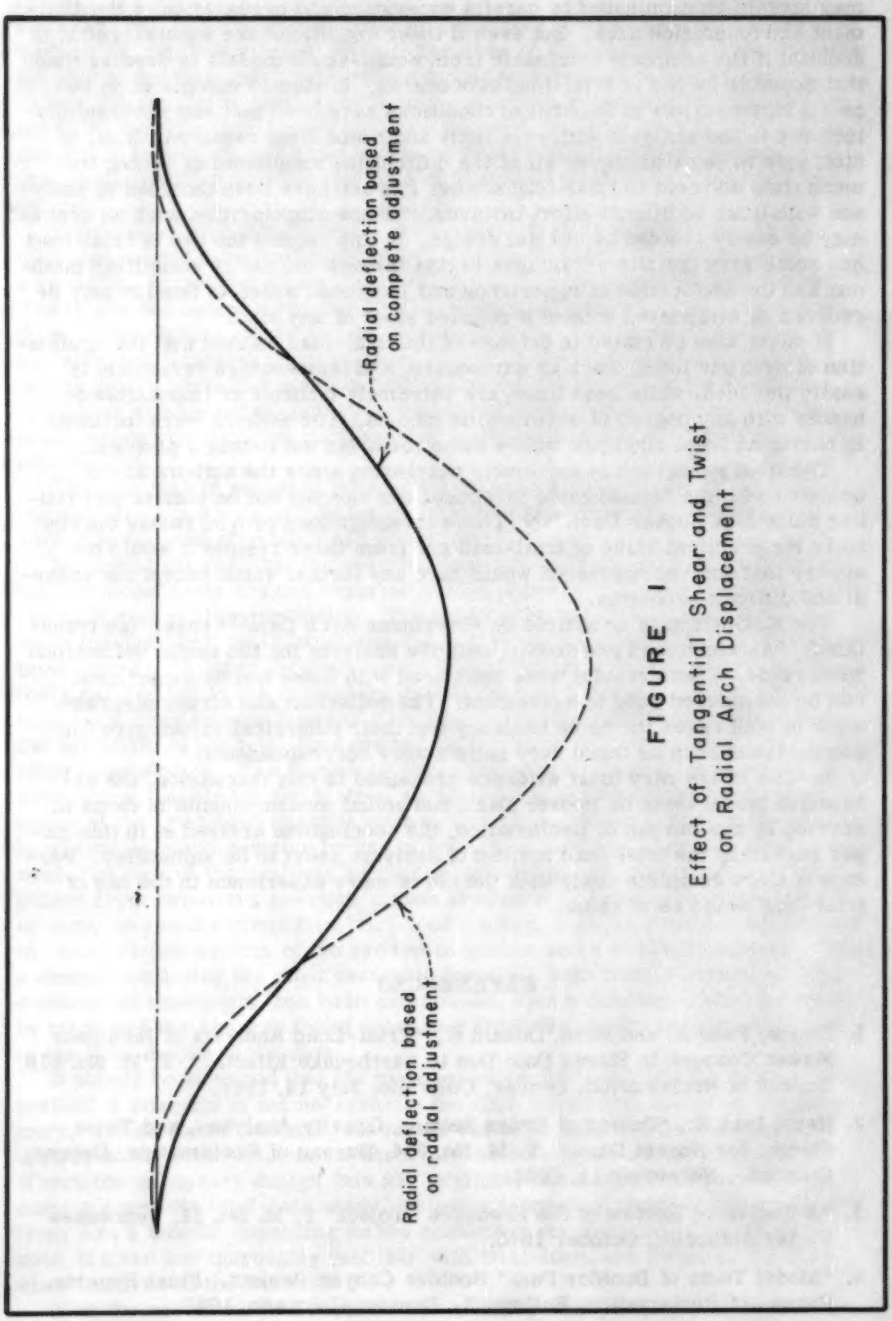


FIGURE 1

Effect of Tangential Shear and Twist on Radial Arch Displacement

TABLE I--EFFECT OF TANGENTIAL SHEAR AND TWIST ON ARCH STRESSES AT CROWN AND ABUTMENT SECTIONS OF YELLOWTAIL DAM

| Elevation | Face of Arch | Crown of Arch | | | Abutment of Arch | | |
|---|---|---|---|---|---|---|---|
| | | Radial Adjustment | Complete Adjustment | Effect of Tang. Shear and Twist | Radial Adjustment | Complete Adjustment | Effect of Tang. Shear and Twist |
| 3660 | E | 622 | 497 | -125 | 543 | 158 | -385 |
| | I | 262 | 350 | +88 | 342 | 138 | -204 |
| 3600 | E | 793 | 685 | -108 | 443 | 139 | -304 |
| | I | 192 | 340 | +148 | 250 | 317 | +67 |
| 3550 | E | 874 | 732 | -142 | 386 | 225 | -161 |
| | I | 135 | 324 | +189 | 295 | 366 | +71 |
| 3500 | E | 900 | 725 | -175 | 284 | 202 | -82 |
| | I | 121 | 258 | +137 | 435 | 488 | +53 |
| 3450 | E | 886 | 705 | -181 | 135 | 137 | +2 |
| | I | 61 | 198 | +137 | 551 | 566 | +15 |
| 3400 | E | 832 | 666 | -166 | -15 | 104 | +119 |
| | I | 58 | 101 | +43 | 675 | 691 | +16 |
| 3350 | E | 765 | 568 | -197 | -106 | 42 | +148 |
| | I | -2 | 93 | +95 | 717 | 778 | +61 |
| 3300 | E | 650 | 521 | -129 | -114 | -4 | +110 |
| | I | -44 | 41 | +85 | 619 | 696 | +77 |
| 3250 | E | 511 | 416 | -95 | -97 | 21 | +118 |
| | I | -49 | 5 | +54 | 504 | 520 | +16 |
| 3200 | E | 377 | 277 | -100 | -30 | 149 | +179 |
| | I | 18 | 17 | -1 | 383 | 413 | +30 |

Note: All stresses are in pounds per square inch. Tensile stresses are indicated by a negative sign. E and I are extrados and intrados of arch.

5. Goriupp, Von K., "Die Berechnung der Gewolbemauer am Hierzmann nach dem Versuchslastverfahren" (Analysis of Hierzmann Arch Dam by the Trial-Load Method) Osterreichische Bauzeitschrift, Vienna, 1950.

TABLE II--EFFECT OF TANGENTIAL SHEAR AND TWIST ON CROWN CANTILEVER STRESSES IN YELLOWTAIL DAM

| Elevation | Face of Dam | Radial Adjustment | Complete Adjustment | Effect of Tang. Shear and Twist |
|---|---|---|---|---|
| 3600 | U | 135 | 132 | -3 |
| | D | -29 | -26 | +3 |
| 3550 | U | 199 | 209 | +10 |
| | D | -20 | -30 | -10 |
| 3500 | U | 209 | 241 | +32 |
| | D | 37 | 2 | -35 |
| 3450 | U | 185 | 250 | +65 |
| | D | 129 | 58 | -71 |
| 3400 | U | 137 | 243 | +106 |
| | D | 251 | 130 | -121 |
| 3350 | U | 73 | 225 | +152 |
| | D | 387 | 212 | -175 |
| 3300 | U | 1 | 196 | +195 |
| | D | 534 | 309 | -225 |
| 3250 | U | -80 | 152 | +232 |
| | D | 693 | 423 | -270 |
| 3200 | U | -170 | 93 | +263 |
| | D | 865 | 557 | -308 |
| 3140 | U | -292 | -7 | +285 |
| | D | 1,091 | 755 | -336 |

Note: All stresses are in pounds per square inch. Tensile stresses are indicated by a negative sign. U and D are upstream and downstream faces of cantilever.

**A. WARREN SIMONDS,[1] M. ASCE.**—The Laboratório Nacional de Engenharia Civil of Portugal has been very fortunate to have had the opportunity of collaborating with Direccao Geral dos Servicos Hydráulicos and with Hidro-Eléctrica do Zézere in the extensive investigation of an arch dam by model tests, analytical computation, and observation. Such an opportunity of having a model and prototype available for research is indeed rare and the results obtained are enlightening. In extremely thin arch dams, cracking of the concrete develops frequently which complicates the analytical computations and also increases the difficulties in similitude for model tests. The fact that cracks developed in the prototype and were reproduced in the models and successful strain measurements made, indicate that models may be used advantageously to provide information on the structural behavior of arch dams and in predicting their behavior.

The tests of both the cofferdam at Costelo do Bode and also its model were made under almost ideal conditions. For the prototype, the loading-unloading cycle was made during a fairly short period of time. This minimized the possibility of having the structure affected by temperature changes, or changes in the elastic properties or creep in the concrete. For the model tests, the

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laboratory conditions could be held constant throughout the test period thereby eliminating many of the uncertainties traceable to changes in temperature and humidity. By eliminating these changes, simpler interpretation of the test results could be made.

Prior to the design and construction of Hoover Dam, the Bureau of Reclamation undertook a program of investigation of structural behavior of arch dams. The Bureau contributed to the test program of the Arch Dam Committee of the Engineering Foundation for the tests of the Stevenson Creek Dam and subsequently built and tested a model of this dam. This was followed by tests of models of Gibson and Hoover Dams. At the prototype of these two latter models, extensive programs of structural behavior were undertaken. It should be emphasized that at the time these investigations were started in the late 1920's, the equipment for measuring strains, stresses and deflections was still in the development stage and the results obtained were crude compared with those obtained by the Laboratório Nacional de Engenharia Civil using the modern equipment now available for this purpose.

As a part of the program of research investigating the structural behavior of Hoover Dam, two structural models were built and tested. This program included measurement of deflection in radial and tangential directions, angular displacements, foundation deformation, and measurement of strain. After making observations of the behavior of the models, analyses of the models were made for the loading conditions in the laboratory.

The first model of Hoover Dam was made of a mixture of commercial building plaster and celite (infusorial earth) which was a highly elastic lightweight material. The principal items of structural data, and the assumptions that were made for the purpose of the analysis, are listed as follows:

1. Scale of Model 1 to 240, or 1 inch equals 20 feet
2. Loading fluid: Mercury having a density of 13.6
3. Weight of plaster celite material 42 lb per cubic foot
4. Modulus of elasticity of the plaster and celite material in tension and compression, 90,000 psi
5. Modulus of elasticity of the plaster and celite in shear, 37,500 psi
6. Normal stresses vary as a straight line from the upstream face to the downstream face of the model
7. Cantilever stresses at the faces of the model acting parallel to the slopes of the faces are equal to the calculated vertical stresses multiplied by the squares of the secants of the angles between the faces and the vertical directions
8. The foundation deformation was adjusted and made to conform to the model measurements
9. The concrete support around the plaster celite base and the support at the downstream side were both assumed rigid

The trial load method was used in the analysis of the plaster celite model of Hoover Dam with a mercury pressure load applied at the upstream face. The strains, stresses, rotations and deflections, due to the mercury load, were determined by the analysis; and since the actual strains and deflections had been measured on the model, the results gave a comparison between the computed and measured deflections.

For this purpose of the analysis, the model dam was replaced by two systems of elements, a system of vertical cantilevers, and a system of horizontal arches. The mercury load was divided between arches and cantilevers,

and also equal and opposite tangential and twist loads applied to the two systems, so that geometrical continuity would be established throughout the structure. The required loads were gradually determined by trial so that the deflections and rotations in each system would be equal at all conjugate points. Since this satisfied the conditions of geometric continuity, the strains and stresses due to the applied trial loads were calculated for both systems of elements.

The mercury load at the upstream face of the model produced movements in radial directions, movements in tangential directions, and rotations in horizontal planes. These movements and rotations were considered respectively in the radial, tangential and twist adjustments. In the radial adjustment, the loads that were necessary to produce radial continuity were applied by trial to the arches and cantilevers. In the tangential adjustment, the loads were introduced that were needed to restore tangential continuity; and in the twist adjustment, the loads were introduced that were required to produce the same rotations in the arches and cantilevers.

The results of the series of tests on the plaster-celite model were that the computed deflections agreed very closely with the measured deflections. The results of the strain measurements were not in as close an agreement, however. This was largely due to the fact that the available gages for measuring strain were very crude compared with the strain gages presently available. The strain measurements were made only on the downstream face of the model because the load was applied by hydrostatic pressure of mercury in a rubber bag braced against the upstream face.

The second model of Hoover Dam tested was made of a compound of rubber and litharge which had a unit weight of 150 pounds per cubic foot. The material had a modulus of elasticity sufficiently low as to deform readily under hydrostatic pressure of water. Thus it was possible to have the specific gravities of the model and the prototype the same, and also that of the loading mediums the same.

The principal items of structural data, and the assumptions that were made for the purpose of analysis of this model were as follows:

1. Scale of the Model 1 to 180, or 1 inch equals 15 feet
2. Weight per cubic foot of water 62.5 pounds
3. Weight per cubic foot of rubber and litharge 150 pounds
4. Average modulus of elasticity in the rubber and litharge material for tension and compression in the cantilevers, 343 lb per square inch
5. Average modulus of elasticity of the rubber and litharge for tension and compression in the arches 389 lb per square inch
6. Average modulus of elasticity of the rubber and litharge for shear in the cantilevers, 114.33 lb per square inch
7. Average modulus of elasticity of the rubber and litharge for shear in the arches, 129.67 lb per square inch
8. Average value of Poisson's ratio 0.5
9. Normal arch and cantilever stresses vary as a straight line from the upstream face to the downstream face of the model
10. The foundation movements were adjusted and made to conform with the model measurements
11. The concrete testing pit supporting the model and its foundation was assumed to be rigid
12. It was assumed that the rubber remained in contact with the concrete of the testing pit at all points

In the trial load analysis of this model, it was necessary to assume that the rubber and litharge compound was a homogeneous and isotropic material. It was not an ideal material, however, because it had pronounced directional properties, which with an abnormally large Poisson's ratio and a proportionally high tensile strength, made its elastic behavior different from that of concrete. However, it did permit making observations for strains on the upstream face when the model was subjected to water load.

The results of the tests of both of these models were compared with the analytical results made using the trial load method of analysis. A comparison of the measured and computed deflections showed that the analysis of the models by the trial load method gave deflections very close to the actual values found from measurements made on the models. The computed stresses were in reasonably close agreement with those obtained from the strain measurements on the models. If the improved models of strain gages had been available at the time of these tests, it is expected that the agreement would have been closer.

The concluding step of the investigation of stresses in Hoover Dam has not been completed at this time. Provisions for measuring the deflection and an extensive installation of embedded instruments, strain meters, joint meters, and thermometers, have been made at Hoover Dam for the purpose of observing the structural behavior of the prototype. It is hoped that eventually a series of observations can be obtained under conditions which will resemble those conditions under which the models were tested. The models were tested under maximum reservoir level and uniform temperature conditions. At the prototype the reservoir level has not reached the maximum and the temperature of the concrete has increased above that which existed at the time the contraction joints in the dam were grouted.

In comparing the stresses obtained from strain measurements at Castelo do Bode cofferdam with those computed by the trial load method, the results are in reasonably good agreement considering how the analysis was made. The deflections, however, were not in as close an agreement as the authors point out. This is probably due to the fact that only radial displacements were considered, the arch elements were assumed to be of uniform thickness, and the modulus of elasticity of the foundation was assumed to be the same as that of the dam. It is expected that a closer agreement of stress results would have been obtained if the trial load analysis had included the effects of the nonuniform thickness of the arch elements, and the tangential and twist adjustments. It is possible also that if the modulus of elasticity of the foundation rock had been different from that which was assumed it would have affected the computed deflections.

JOE T. RICHARDSON,[1] A.M. ASCE.—This paper is a valuable addition to the slowly accumulating published results of stress studies made on actual in-service dam structures, along with allied model tests. The authors are to be congratulated for their splendid presentation of results from several independent measurement sources and the comprehensive manner in which the correlated data are prepared.

Few persons will probably realize the amount of time that is represented in the results of these studies. Endless man-hours of work for a large

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number of persons are represented in the accumulating of data from model and prototype sources, correlated laboratory studies, computations required for each study, and the preparation of the plotted results, not to mention the amount of time required for the trial-load and other analytical studies represented. The organizations represented by these authors are to be complimented for the conducting of this program. This summary is one of the first for which measured stress results are shown for an arch dam; and it serves to establish, along with analogous gravity dam results,[2] information by which the engineer will have some criteria that will serve to gage methods of design.

It is not surprising that model and prototype results and the results of trial-load and other analytical studies are not comparable. It must be realized that studies of stress by measurement and studies of stress by analytical methods are each on a basis different from the other; and each reflects the assumptions and conditions for which the particular analysis is made; and thus, cannot be expected to show the same final results. Perhaps the authors of this paper should have omitted results from their trial-load and other analytical studies at this time. Since no attempt has been made to point out reasons for the differences existing between trial-load and other analytical results and the results of the model and prototype studies, the comparison of stress as presented is somewhat misleading and is not made in all fairness to the trial-load method. The trial-load studies were indicated by the authors to have been made only on the basis of a radial adjustment and do not include the effects of tangential shear and twist, as well as varying arch thickness. In contrast, Bureau of Reclamation practice has been to make an initial study based on radial adjustment, then to complete the study by including the effects of tangential shear and twist. In all cases the varying thickness of the arch is considered. The comparison of an initial and complete trial-load study shows quite different results. The authors' comparison, using trial-load results that disregard these important factors, is not understood.

The conditions of temperature as affected by factors of progressing construction, as well as the varying conditions of reservoir, are reflected in the results of instrumental studies and are not easily represented in the analytical studies, and are not represented to the same degree in the model studies. Oftentimes, the analytical study represents only stress conditions during a normal or some other fixed operation condition for the structure, whereas in-service conditions for a structure may not ever be comparable, since loading, as represented in the analytical study, may not occur on the prototype for a number of years after completion of the structure. Even then, stress conditions may be entirely different, due to the wide difference in temperature conditions and deflection of the dam between actual conditions and those used in the design. The analytical study must assume values for factors, such as allowable concrete strength, modulus of elasticity, temperature effect, and Poisson's ratio; and oftentimes does not recognize the effects of factors, such as concrete growth or shrinkage due to drying or creep of concrete under sustained load. Likewise, the effect on stress of triaxial loading of concrete is not included. The triaxial effect on stress of volumes of concrete, as confined in a massive concrete dam, are not too well understood,

2. J. M. Raphael, M. ASCE, "The Development of Stresses in Shasta Dam," Transactions, ASCE, Paper 2547, Vol. 118, 1953.

and thus cannot be included in the analytical study. These quantities are more or less automatically introduced, or at least accounted for to some extent, in the measurement studies. Rock modulus is a more or less uncertain quantity. Although modulus values as used in analytical studies are determined to the best of the ability of the laboratory worker, using accepted methods, the modulus may often be only relative when thought of from the view and magnitude of abutment and foundation rock mass, considering the enormous volume of foundation material surrounding a dam and the varying effect the dam and reservoir mass may have on that foundation rock mass.

Considering all of these factors collectively, it can easily be understood that many designers lose sight of the complex system of variables with which they are dealing in an arch dam, and oftentimes are reluctant to accept results of measurement methods of analysis when these results are not in agreement with the results of analytical methods. Conversely, many research workers do not want to accept results obtained by analytical means when these results do not show the same thing derived from model and other measurement studies. In general, when a comparison is made, results must be qualified to avoid misleading the reader who may not be familiar with the basis of the study or the method used.

Many designers and research workers are of the opinion that stress variation on a section of a cantilever or arch is of a linear nature, as expressed by accepted theoretical and mathematical methods, and are unwilling to recognize that stress variation obtained through measured strain is indicated to be very much nonlinear in nature, as shown by results of measurement studies, not recognizing that stress is a function of varying temperature as well as load. Sometimes, temperature effect on a massive structure is of far greater influence on stress results than that of load.

Only through results such as presented by this paper are these facts made known, and only by the continuing joint efforts of persons connected with both measurement and analytical programs can sufficient information be accumulated that will be useful to the engineer as a means to effect the narrowing of the gap that exists between design and in-service conditions for a structure. These results from measurement and the associated economies that can be effected through recognition of conditions resulting from these studies, along with analytical studies for design, will assure that future as-built structures will operate stresswise as anticipated by the design.

In an analytical study, the resulting stresses are somewhat an average stress resulting from the combination of loads, moments, shear, and twists that are applied through methods of the analysis. Considering a structure as a series of vertical elements analogous to posts or pilings, and computing stress at required horizontal sections will result in an entirely different stress pattern on the vertical plane than that usually used in an analysis, particularly near the base of the structure. This effect was illustrated in the results of a previous paper[2] and is further strengthened by other similar studies. The results of the in-service studies from measurement and the results of trial-load studies show, in some cases, large differences. This post or piling explanation may partially account for these differences between the results of the measured stress, which may be termed "observed" stress, and the results of the computed stress, which may be termed "average" stress.

2. Ibid. reference 1, p 308.

The authors will be interested to learn of the usefulness and reliability of external measurements as a check on stress trends shown by strain meters embedded in an arch dam. In the recently completed study of stress from measured strain in the Hoover Dam of the Bureau of Reclamation, the stress trend in both cantilever and arch sections from strain meter measurements is reflected in the record from the plumb lines located in the elevator shafts and from the precise survey measurements made over the top of the dam.

From the program presented by the authors of the paper, it is apparent that model and prototype structural studies provide as authentic and useful a source of data for evaluating the behavior of a structure as is provided by analytical studies. While the trial-load and the independent arch methods of analysis furnish an excellent beginning for determining stress conditions to expect from a structure, they still do not guarantee that the structure will behave in accord with results of the respective stress studies. Thus, the model and prototype results are of great values for the evaluation of the merits of the method and for use as criteria in future structures, and in the end an attempt should be made to tie results of all three methods together.

The authors' statement regarding behavior measurements at Bureau of Reclamation and Tennessee Valley Authority dams is not entirely clear in the following sentence:

> "It should be noted, however, that in all of those investigations no attempt was made to analyze the stresses by computation or model testing."

The authors of the paper are reminded that Bureau of Reclamation structures have been analyzed by one or several methods. In the large concrete gravity arch dams, analytical studies by one of several methods are made as well as study of stress by internal and external measurements. In the case of Hoover Dam, an elaborate program of model testing[3] was carried out simultaneously with the trial-load and other analytical studies. At this late date, stress from measured strain correlated with measurements of deflections and displacements, will be made available within a short time. An illustration that will be of interest to the authors of this paper, showing comparable results between strain meter studies and theoretical nonlinear stress studies, is offered in the following example from the recently completed strain meter study of Grand Coulee Dam (Fig. A).

It will be noted that for the results of each study the stresses for the most part are in very good agreement.

In the prototype example offered by the authors, it is noted that the loading-unloading cycle for the structure required only 4 days, which is far from conditions experienced in most service structures. No mention is made regarding the method of application of the property of creep of concrete under sustained load and its treatment in relation to the prototype study. Thus, it must be assumed during this short loading cycle that for the purpose of simplicity of solution, creep of concrete may be neglected and only a creep modulus of elasticity at time of full load used with the resulting assumption that stress will follow directly from strain through direct application of Hooke's law.

The ingenious orientation of strain meters placed in the prototype is

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3. Boulder Canyon Final Reports, Part V, Tech. Investigations, Bull. 3, Model Tests of Boulder Dam, and Bull. 6, Model Tests of Arch and Cantilever Elements.

noted. Here, the strain meters are placed in trios, equilaterally, in planes parallel to the faces of the dam. While this orientation is excellent from the aspect of economy, it affords no means for a check on strain at the gage points as would be assured by a duplicate meter installation. Thus, measured strain values must be considered absolute, and strain for directions other than those of the meters must be computed by resolution. Though the authors may have had implicit faith in the reliability of their equipment and the methods used for installation, it is pointed out that, had any one of a trio failed, the complete trio would have been useless; also, without duplicate meters in each gage direction, there always exists the shadow of doubt regarding readings. This scheme of meter installation is in contrast to that used by the Bureau of Reclamation. Bureau practice provides for duplicate strain meters to be installed in the three major orthogonal planes and

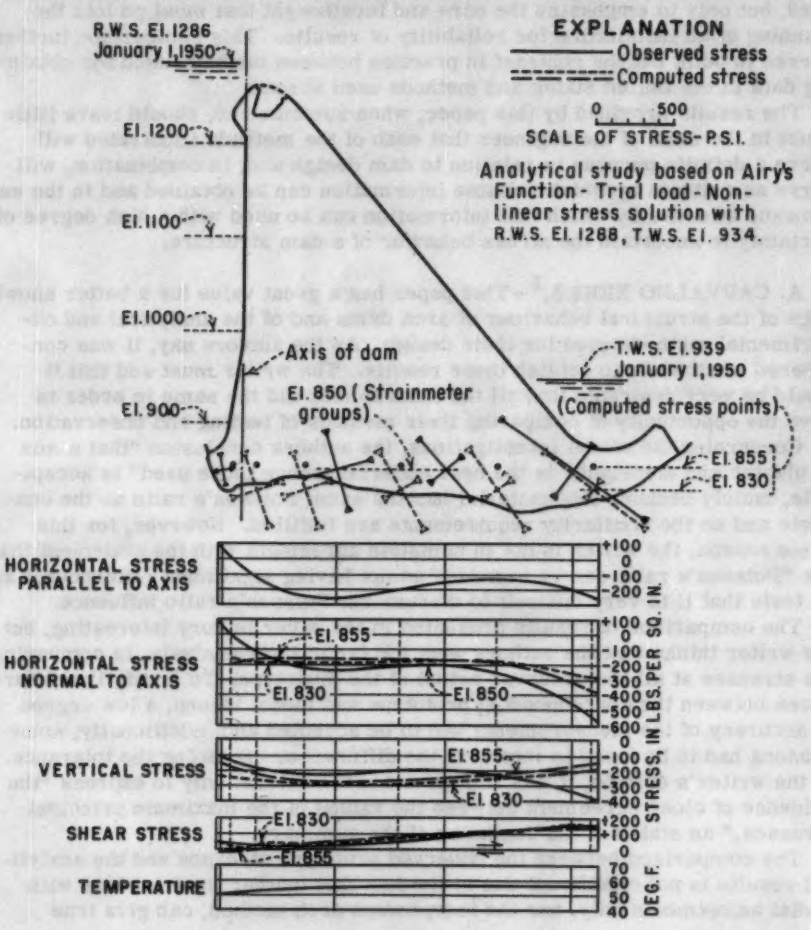


Fig. A. Comparison of Observed and Computed Stress - Grand Coulee Dam - Maximum Spillway Section

supplementary meters placed in several planes at 45° angles, with the meters of the primary planes. The usual installation includes 12 meters for each group at a point of measurement. Thus, the multi-directional gage line system affords three major directional lines of stress from strain at the gage point and by addition of extra meters in midangle positions affords the greatest possibility and reliability for stress investigation at a gage point. This scheme affords assurance of better compatibility of strain between trios of strain meters having one mutually perpendicular meter in the trios and adds to the reliability of results gained from the installation. In the installation made by the authors in the prototype, no strain meters were installed centrally in the sections of the dam between the upstream and downstream faces, thus results for stress variation through the section are unknown.

These comments are not to be construed as a critical view of the system used, but only to emphasize the care and forethought that must go into the planning of an installation for reliability of results. This comparison further serves to point out the contrast in practice between methods used for obtaining data in the United States and methods used abroad.

The results provided by this paper, when summarized, should leave little doubt in the mind of the engineer that each of the methods illustrated will serve a definite purpose in relation to dam design and, in combination, will serve as criteria by which reliable information can be obtained and in the end a means afforded by which that information can be used with a high degree of certainty to ascertain the stress behavior of a dam structure.

A. CARVALHO XEREZ,[1]—This paper has a great value for a better knowledge of the structural behaviour of arch dams and of the analytical and experimental methods used for their design. As the authors say, it was considered worthwhile to publish these results. The writer must add that it would be very desirable that all the laboratories did the same in order to have the opportunity of comparing their methods of testing and observation.

Concerning the model investigations, the authors conclusion "that a mix of plaster and kieselguhr is the best material among those used" is acceptable, mainly because this material has the same Poisson's ratio as the concrete and so the similarity requirements are fulfilled. However, for this same reason, the writer is not in complete agreement with the statement that the "Poisson's ratio can be regarded as not having significant influences," as he feels that it is very difficult to discuss the Poisson's ratio influence.

The comparison of results presented in the paper is very interesting, but the writer thinks that the authors went too far in their analysis, in comparing the stresses at all the observed points of the prototype. To justify the differences between the corresponding prototype and model values, a low degree of accuracy of the measurements had to be accepted and, additionally, some reasons had to be found to interpret the differences exceeding the tolerance. In the writer's opinion, it was enough and more natural only to express "the evidence of close agreement between the values of the maximum principal stresses," as stated at the beginning of the summary.

The comparison between the observed prototype stresses and the analytical results is not significant due to the fact that neither the trial load with radial adjustments only, nor the independent arch method, can give true

1. Technical Director, Hidro Eléctrica do Zêzere, Portugal.

results for this type of dam. The second method has the great value of its simplicity, giving the superior limits of the principal compressive stresses.

Finally, it must be stated that the conclusion that "an upstream crack in the base of the dam does not sensibly change the state of stress, even though it affects an important part of the dam thickness" is very important. In fact, this conclusion is very helpful for the designers of dams.

Discussion of
"SELECTION OF INSTALLED CAPACITY
AT HYDROELECTRIC POWER PLANTS"

by Lesher S. Wing and Robert H. Griffen
(Proc. Paper 697)

ROBERT F. KRAFFT,[1] A.M. ASCE.—Selection of the proper installed capacity at one or more potential hydroelectric power sites has been approached in various ways, some overly simple, others overly complex. The authors have conferred a distinct service on hydroelectric planning engineers by presenting the basic concepts involved and an orderly procedure to facilitate the solution of such problems. The basic problem is to "supply the system load at the lowest cost" at any time as power demand increases. Such lowest cost may, at different times, be obtained from all hydro, all steam or various combinations of these.

Many assumptions, rightly, have been made by the authors for the purpose of simplification and clarity. However, some of these assumptions may lead to erroneous conclusions unless they are fully understood and consideration is given to their effect.

Prime Power vs Firm Power

Prime power is available continuously in the same amount at all times. Such power by itself is not fully marketable. A hydro plant developed for prime power only must be integrated with other plants, hydro or steam, to serve system load, or some prime energy must be wasted. If the installation at the hydro site is increased in order to convert prime power and prime energy to firm power and firm energy, some secondary energy usually is made available. This secondary energy also may be made marketable by integration with other plants, particularly steam plants. A more definite measure of the potentialities of a hydro site would be to determine the maximum amount of firm power under adverse conditions. All additional potential energy would indeed be secondary.

Elimination of the Prime Power concept would simplify and clarify the procedure and would not discriminate against any economic development. It would be a very unusual hydro site where the relationships were:

1) Prime Hydro Cost equal to or less than Steam Cost, and
2) Net cost of incremental capacity from Prime to Firm greater than Steam Cost.

Under this concept, each plant in a system would have the obligation of supplying a proportionate share of load. If and when it could not do this, supplemental power would have to be obtained from some other source. If and when secondary energy was available, a market would be sought in the load that some other plant was obligated to supply.

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Cost and Value of Power

The authors, rightly, evaluate hydro power delivered at market as being equal to the cost of steam power at the same location; express the value in terms of the two components, capacity and energy; and state that the fixed and operating costs used should be those applicable to the construction agency because erroneous conclusions would result, for example, if tax-free financing were used for hydro and not for steam. However, the authors, for simplicity, assume all power delivered at a "load center" rather than, possibly, to several load areas and purposely omit consideration of reserve capacity, considering reserves to constitute a separate problem.

Reserve Capacity

Reserve capacity may not be an entirely separate problem. Reserves in capacity and energy are required for several purposes, the most important being:

1) Rolling reserve immediately available to meet (a) rapid changes in load or (b) generation or transmission failures.
2) Outage reserve available within reasonable time to replace the rolling reserve.
3) Overhaul reserve available to meet scheduled outages.
4) Reserve for adverse water supply conditions.
5) Reserve for load growth in excess of that estimated.

Either hydro or steam can act as reserve but hydro, especially impulse type units, are more suitable for rolling reserve when not restricted by water supply and steam units are more suitable for other reserves if they can be shut down or backed off to save fuel when not required.

Even in Phase I of system development, steam plants may be economical, if not essential, to standby for failure of transmission lines from hydro sources to important load centers. If located at the end of long transmission lines, steam units can be designed to act as synchronous condensers. Being available, such steam units also can supply supplemental energy in the event of adverse water supply conditions and to carry load as required.

As the ratio of steam to hydro is increased in Phase II, generation by steam units can be reduced during off-peak hours to afford a market for secondary hydro energy, if available. However, the amount of such reduction may be limited by such factors as:

1) Transmission limitation to cities or areas at great distance from hydro sources. It may be more economical to use fuel even when potential hydro energy is wasting than to construct transmission lines used only for relatively short periods.
2) Steam plant design. Unless specially designed, with consequent greater cost, loss of efficiency, or both, steam units must be operated at some load greater than that required for station use in order to be available for quick pick-up.
3) Steam units may be constructed in conjunction with industrial or oil refinery plants, in which case relatively large base load must be carried to supply process steam.

Also, economical planning will require hydro plants to act as reserve for steam outages, especially scheduled outages for overhaul. Generally speaking, steam units require overhaul more frequently and are out of service for

longer periods than hydro units. Consequently, as the steam ratio is increased, the percentage reserve for overhaul must be increased. If system peak load demand varies during the year, it will be economical to schedule overhaul periods of lesser demand. This results in the requirement that hydro capacity should be nearly constant throughout the year at greater capacity factor than might be concluded from a study which did not consider reserves.

Average Usable Power Supply

Potential power from a hydro plant or site varies greatly between years and seasons dependent upon water supply. The energy available in a wet year usually will far exceed the average available over a period of years and, unless the system has a very low ratio of hydro to steam, some potential output will be wasted. As the ratio of hydro to steam decreases, some of this nonsalable power can obtain a market. The existence of such potentially available energy from existing sources should be considered in estimating the amount of marketable energy available from a proposed hydro development. This consideration may require a considerable discount of potential hydro energy.

Adverse Power Supply

The authors, rightly, have based system planning on the hydro supply available under most adverse conditions plus the supply available from steam plants. They have, for simplicity, assumed that the system steam plants are capable of operation at 100 percent capacity factor during such time. The duration of the adverse period is extremely important. For a period of a few weeks such operation of steam plants would be practicable. Such operation for two or three months might be considered reasonably possible, but for longer periods it is believed that dependence on operation greater than 85 percent capacity factor is too optimistic. If the adverse period continues a year or more, as would be the case with cyclic storage, provision for overhaul outage should be made, thus reducing the overall capacity factor to about 75 percent.

System Load Curves

Correct delineation of system load curves is a tedious process. Demand for power varies from hour to hour, day to day, month to month and year to year, dependent upon many factors, including weather, economic conditions, competing sources of power, etc. In an area where irrigation water is pumped, system load increases in dry years when adverse hydro power supply also occurs. Similarly, air conditioning will increase system load in hot summers and space heating will increase system load in cold winters when water supply may be reduced by freeze-ups in the mountains.

At least two sets of system load curves should be developed: one for maximum power demand, to be used in conjunction with adverse hydro power supply conditions for planning installations to supply demand and furnish reserve capacity; the second for minimum power demand, to be used in conjunction with wet year power supply to determine the amount of waste of potential hydro energy.

Low Load Factor Hydro

The upper portion of a system load curve indicates that a portion of operating plant is required at very low capacity factor. In general, and especially under conditions of adverse hydro power supply, this position on the load

curve should be allocated to plants capable of great flexibility. Such plants also may be furnishing rolling reserve for the system at the same, or at other times. The necessity for operation on the extreme system peak cannot be forecast with a reasonable degree of accuracy. The demand may occur one day in a week or possibly five days. It may continue for a few minutes or a few hours. The water used for such a purpose cannot be dedicated to any other purpose, such as irrigation or supply for a chain of plants, unless adequate re-regulation storage intervenes.

## CONCLUSION

The procedure proposed by the authors, modified to deal with firm power rather than prime power, will be found very useful by hydroelectric planning engineers. However, it is recommended that caution by exercised in the determination of maximum and minimum capacity factors of marketable hydro power. Each system or region has its own pecularities as to load factors, seasonal variation, transmission limitations, secondary energy available from existing developments, etc. Economic error may well result unless all factors are considered and their effect analyzed.

## Discussion of "PERMEABILITY, PORE PRESSURE AND UPLIFT IN GRAVITY DAMS"

by Roy W. Carlson
(Proc. Paper 700)

J. LAGINHA SERAFIM,[1] A.M. ASCE.—This paper is of great interest in the uplift debate as it deals with important points which have not been sufficiently considered in the various A.S.C.E. publications on the subject.

The writer recently had occasion to consider in great detail(1) the various aspects of uplift in masonry dams, comparing the different opinions expressed in the vast bibliography on this matter. One of the first conclusions resulting from this comparison is that, in fact, there are three distinct problems, and not two, in the discussions of uplift: (1) What is the distribution of pressure of the water in the concrete pores? (2) What is the effective area over which that pore pressure acts? (3) What is the interpretation to be given to the stresses set up by these pressures?

1) As to the first problem the author gives some very useful information when presenting the implications of permeability tests. He shows, for example, that, as the time necessary for the complete saturation of a dam is very great, only in very special cases will there be a linear pressure distribution across a dam. That is, the percolation of the water will not, as a rule, be steady. In an analysis(2) of unsteady flow of water through concrete it was concluded that this phenomenon is expressed by a differential equation of the type

$$K\left(\frac{\partial^2 p}{\partial x^2}+\frac{\partial^2 p}{\partial y^2}+\frac{\partial^2 p}{\partial z^2}\right)=\left[C_1+C_2\frac{pa}{(p+pa)^2}\right]\frac{\partial p}{\partial t}+C_3\frac{\partial \theta}{\partial t},$$

in which $K$ is the coefficient of permeability, $p$ is the pore pressure, $t$ the time, $pa$ the atmospheric pressure, $C_1$, $C_2$ and $C_3$ constants which depend on the physical properties of the concrete and of the quantity of air trapped within the concrete pores during the filtration of the water and $\theta$ the sum of the three principal stresses, $\theta = \delta_x + \delta_y + \delta_z$, which vary with $p$.

The similarity of this equation with Fourier's equation for heat conduction shows that the interpretation made by the T.V.A. of the very interesting pore pressure observations in Hiwassee Dam is justifiable.(3)

With regard to the remarks made about capillarity, the writer by means of very simple equipment determined capillary rises of about 50 ft for concretes made in the laboratory.(4) This equipment was used first for permeability tests with a view to verifying the validity of Darcy's law for the flow of water under low head.

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2) The problem of the effective area of porous bodies is closely related to the problem of the interpretation of the stresses set up by the pressures in the pores. Hence it is convenient to discuss the latter first. In fact the writer believes that the lack of due consideration of this third problem is one of the reasons for the appearance of so many differences of opinion about the forces produced by pore pressure. Certain statements of the author, such as ". . . the stress distribution is the same when uplift pressure is acting as when there is no pore pressure." or ". . . the uplift force on any section depends only upon the pore pressure and area at that section, . . ." or again ". . . the uplift is produced locally by the pore pressure acting as a jack within the pores." can be understood better if the third problem is discussed. That is, it is necessary to give acceptable interpretations to the terms "uplift" and "stress."

There is nothing new in what follows. Publications like those by Hoffman, Terzaghi and Brahtz(5) clarify the meanings of these terms. On the one hand when we speak of stresses in concrete we refer to the mean stresses, that is, their mean values in areas considerably greater than those of the pores or the size of the aggregates. When speaking of uplift stresses the same meaning of the term stress has to be maintained. But, on the other hand, when dealing with forces produced by pore pressures, the scale of dimensions has to be reduced to the size of these pores and the size of solid bonds between grains.

If a porous body such as a concrete cylinder be submitted, externally and in the pores, to a uniform pressure, it will not break and the deformations produced in it will be negligible.[2] The well known breaking tests by tension (Fillunger) and by compression (Terzaghi and Rendulic and Bureau of Reclamation(6)) on samples submitted to uniform pressures in the pores and on the outside, show clearly (and it is doubtful whether they show more than this) that the strength of concrete, both tensile and compressive, is not altered by high pressures in the pores. In view of the above it can be stated that a body subjected to a uniform pressure produced by a fluid acting all around the entire surface of the solid material is not subjected to uplift forces; however, pressures are acting in its pores. In other words, it can be said that the "total stresses" in such a body are at all points equal to the pressures in its pores and that the "effective stresses" (those which rupture or deform) are nil. The expression "effective stress" can have for concrete a meaning analagous to that given in Soil Mechanics.

Once the rupture of concrete in tension or compression appears to result from the rupture of the bonds between the solid particles,(7) in sections with a minimum area of solid, it is only of interest to consider the effective stresses in these sections. Note that such sections are simultaneously those of the greatest "effective area." Also it is the deformations of the solid bonds which principally account for the deformations of the concrete when it is loaded by external loads.[2] So, the effective stress is, for soils, an indication of the contact forces between the grains and, for concrete, an indication of

2. The writer estimated by means of tests(1) that the value of the compressibility of the solid material of concrete corresponds to a bulk modulus of elasticity in the order of 11x106 psi, to which a value of the modulus of elasticity for the solid material of $17 \times 10^6$ psi corresponds approximately.

the forces transmitted by the solid bonds in the section of smallest bond area.

Now imagine a concrete pillar 100 ft high embedded in impermeable rock at the bottom of a lake 200 ft deep. In the pores of the concrete, at the base of the pillar, there would be a water pressure of 86 psi and at the same time the uplift at the base of the pillar (taking the effective area as 100%) would only produce (effective) tensile stresses of 43 psi.

Summing up, the value of the pore pressure does not give, by itself, the value of the stresses due to these pressures. It appears, therefore, more reasonable to admit, as did Fillunger, Hoffman and others, that the uplift forces result from the gradients of pressures in the pores between the point under consideration and the surface of the body.

From the definition adopted above for the effective stresses, it can easily be shown[8] that the forces set up by pore pressures are in the nature of body forces in which the components per unit volume are the derivatives of the pressure p times the effective area n:

$$\frac{\partial np}{\partial x} \; , \; \frac{\partial np}{\partial y} \; , \; \frac{\partial np}{\partial z} \; .$$

Note that, if the effective area is not constant from one point to another, its variation is responsible for the presence of body forces. Hence it can be concluded that the uplift at a point depends on and is produced by the presence of variations in the pore pressure or the effective area between one point and another. But, as in concrete the cement paste, which normally is the material having the greatest effective area, surrounds all the aggregate, n can be considered as constant in the calculation of the stresses which interest the engineer (mean effective stresses).

From the consideration of the body forces referred to above it has been demonstrated[8] that a loading due to pore pressure (steady or unsteady flow) is analogous to the loading produced by temperature variations given by

$$T = \frac{1-2\mu}{\alpha E} n_p \; ;$$

E and $\mu$ being the elastic constants of the concrete and $\alpha$ its coefficient of thermal expansion. It has also been demonstrated that in simple cases (state of plane strain and pressures corresponding to a steady flow), the uplift stresses are given by the difference between the stresses which would act on the body if it were impermeable and the value of the product of the pressures in the pores by the effective area.

3) Let us pass to the second problem. As to the values of the effective area of permeable bodies, it was Terzaghi who stated that it could only be determined by tests. However, the deductions of Mr. Carlson are quite acceptable from the logical point of view and also do not contradict previous experimental results.

Considering various tests, the opinion of the writer is that the value of that area depends on the test conditions; i.e. the effects of the pressures of a fluid in the concrete pores are different according to whether we are dealing with rupture tests or tests by measuring deformations and also according to the type of fluid used in the tests.

In the tests carried out by Leliavsky[9] the samples were tested up to

rupture. This was caused by tensile forces set up by the pressure variations in the pores from the end, where the fluid was coming out, to the zone of failure. The values of the effective area found by this author (mean value 0.91) have therefore to be taken into consideration when it is sought to verify the stability of constructions in relation to rupture.

In tests carried out by the writer (similar to those of Prof. Davis referred to by the author) the object was to measure the forces and deformations produced by pore pressures. Ten series of three or four mortar cylinders and two series of neat cement paste cylinders were tested. Using the equipment shown in the figure a tensile force set up by the longitudinal variation of pressure was obtained. To avoid rupture by tension the cylinders were maintained in constant axial compression during the test. The total longitudinal deformations and also the strains in the mean section were measured in percolation tests with nitrogen and with water. In the tests in which strains were measured, as there was a compensating strain gauge mounted on a small briquette (of the same material as the cylinder) placed in the chamber, it was not necessary to subtract, from the readings obtained, the strains produced by the volumetric compression of the solid material. In fact, these were eliminated in the reading given by the strain indicator. In the tests in which the total longitudinal deformations were measured by external dial gauges, the longitudinal deformations due to the volumetric compressibility of the solid material were deducted from the values obtained.

The tensile forces set up by the percolation of the fluid were calculated from the values of the deformation or strains produced by the longitudinal compression and from the values of the decrease of these deformations or strains caused by the pore pressure. These forces allowed the effective area of the samples to be estimated taking into account the effect of creep.

The nitrogen tests on dry samples produced values of the effective area, $\underline{n}$, varying from 0.8 to 1.0, the results showing that $\underline{n}$ increases slightly with the water cement ratio, with the value of the pore pressures and with the dryness of the samples. The tests where water was used supplied values between 0.40 for compact mortars and 0.80 for lean mixes. In one of the tests, water at a pressure of 560 psi was maintained around the sample during 19 days and there was no increase of $\underline{n}$ after two hours following the application of the water pressure round the sample.

The low values obtained by the water tests suggested the lack of mobility of the water films adsorbed. The nitrogen tests also showed that the area of solid bonds between particles is small compared with the area of the voids.

The results of these tests do not agree with the statements which the author makes before he says that "the test program (of Prof. Davis) could not yield the effective area in the simple manner that was anticipated." Were it so, the values found now would always have been equal to unity. The writer feels very much more inclined to believe in tests rather than in the theoretical considerations even though they may have the interest and logical exactness of those which were presented by the author, for "experiment is the sole source of truth."(10)

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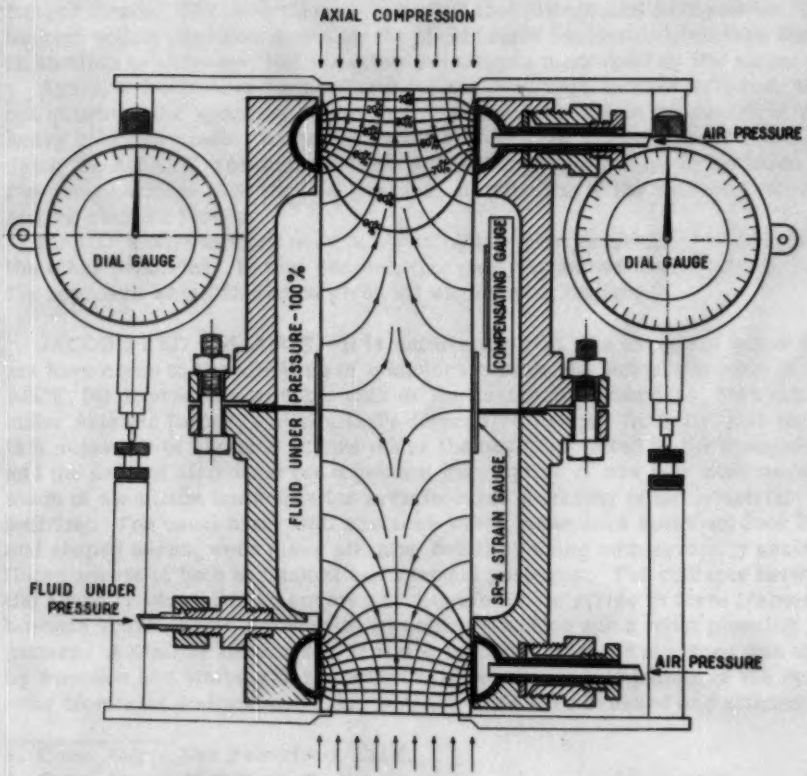


Fig. A

## Discussion of "MORRO BAY STEAM ELECTRIC PLANT"

by J. George Thon and Gordon L. Coltrin
(Proc. Paper 737)

GLENN B. WOODRUFF,[1] M. ASCE.—The authors have given an excellent description of the civil engineering features of a modern steam electric power plant. To the designer of a comparable facility this is less important than is the description of the careful analysis of the various possible alternates which preceded each step in the design. In this latter respect, the paper describes methods of approach permitting considered judgment as contrasted to following precedent or of making "snap" decisions. For illustration, comments on some of these studies follow.

The upper strata are recent alluvial deposits, subject to consolidation under load. Snap judgment would have dictated piles extending to the lower and firmer strata. The investigation indicated that differential settlements could be kept within tolerable limits by the rigid frame foundation structure which, in addition to economy, had the other advantages mentioned by the authors.

Again, it would have been natural for the designers to have followed, without question, the specifications of A.C.I. Committee 505 in the design of the heavy concrete stack. While the writer believes the assumption of C = 0.13, using the A.S.C.E. recommendations somewhat conservative, he believes the resulting sections give the best possible distribution of the material for resisting seismic forces.

Similar analyses were used in securing the most economical solutions of the other problems. In this respect, this paper is an outstanding example of the approach which should be given all engineering designs.

JACOB FELD,[2] M. ASCE.—It is unfortunate that this excellent paper may not have come to the attention of members outside the Power Division of ASCE, for it presents valuable data on the design of foundations, high stacks under seismic forces, and similarly diversified topics. Actually, this paper is a summary of six good papers under the headings listed in the Synopsis and the project also deserves a general description of how new uses were made of aluminum materials for architectural surfacing of an industrial building. The usual blank wall surfaces which make such buildings look like odd shaped barns, were given pleasing detail by using commercially available fluted sheets in both horizontal and vertical positions. The contacts between the areas so obtained are covered by flat aluminum strips to form frames or borders which create differences in color reflection and a most pleasing pattern. A similar method for breaking up large exterior surfaces was used by Ammann and Whitney in the exterior precast concrete panels of the Syracuse Memorial Auditorium. The castings were wire brushed and alternate

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2. Cons. Engr., New York, N. Y.

panels are set with vertical and with horizontal markings.

The interior of this shell contains a plant, novel in many respects, which is certainly of interest to the members of the Power Division. The writer, however, wishes to bring to the attention of the other members some of the valuable data in their fields. Like with many other structures, the control of materials during the construction period is more difficult and opens the greater possibility of non-success, than after the substantial completion of the assembly. One such problem was to prepare the fine dune sand into desired slopes and to keep them in such shape. Anyone who has experienced this type of soil, which is either dust when dry or soup when wet, will appreciate the solution described in the paper. The continuous fine spray control of the moisture content to just the correct amount to prevent wind erosion and at the same time protect against liquification and sloughing, of thirteen acres of surface is no small project. In a somewhat similar problem, but not as large, the writer used a spray of emulsified asphalt into which fertilizer and selected grass seed had been mixed. The asphalt skin aided considerably in holding the surface, to keep the seed where desired and at the same time discouraged feeding by birds. The asphalt in no way retarded germination and actually seemed to act as a delayed fertilizer, once the grass sprouted. Some years later, a commercial product appeared on the market consisting of grass seed coated on newspapers with asphalt spray, with the recommendation that the sheets be pegged down on slopes, seeds on the underside, with the claim that the paper will eventually rot and form a fertilizer.

With population drifting to arid and semi-arid areas, the supply of water becomes more of a controlling problem. The large scale sea water evaporation equipment will be a model for studies in many foreign countries. The problem is really three fold: preparation of potable water which is known to be an expensive matter; water for industrial use where appreciable chemical contents can often be tolerated or compensated; water for agriculture where not enough is yet known as to how far plants can be modified to reduce the necessity for salt removal. With the very heavy plant growth in salt water marshes, it seems possible that food plants can be developed to permit irrigation with sea water from which only a portion of the salinity is removed. This is parallel to the development of metal alloys and bearings which permit the use of sea water for lubrication and cooling, in place of normal equipment which requires potable water to prevent excessive corrosion and maintenance.

The description of foundation designs, estimates of settlement and the record of actual settlements is another valuable contribution. The reasons for the choice of a spread footing mat on the soil as against a piled foundation for the main plant and its heavy equipment are completely given and the result certainly justifies the decision made. However, the mat under the stack is supported on piles. The authors should explain why the opposite decision was made in the latter structure, even though soil conditions seem to have been the same and the loads are comparable. Also with the expected settlements so closely checked by the measured dishing of the main mat, could not at least half be compensated by cambering the mat and reduce the necessity for equipment and piping adjustments, although the scheme of hanging the boilers free of the foundation must have eliminated many adjustment problems. The discussion shall not omit reference to the clear summary of design studies and accepted procedure for the stack under seismic loadings and of the turbine and generator foundation under self-induced vibrations. The writer hopes that the discussion will bring the paper to the attention of many members outside the Power Division.

WILLIAM W. MOORE,[1] M. ASCE.—The authors have presented a most useful description of many unusual design features of the subject plant. These data will no doubt suggest valuable design concepts and be of much aid to designers of other projects. In reference to the foundation features of the Plant, there are two factors which warrant particular notice. The first is the results of frequent collaboration between the design engineers and foundation engineers throughout the design period. The second is the attention given to the effects upon the structural and design details of the settlement patterns expected as a result of the sequence of foundation construction and loading.

It should be noted that very different foundation design criteria were employed for the different Plant structures; that is, a rigid mat structure for the main power building, driven piling for the stacks, and spread foundations for tankage and several other smaller structures. These selections were the result of considering the cost and behavior of each unit individually, and the resulting economy was substantial in comparison with the use of a single foundation design for the entire site. Such results could not be obtained if a foundation exploration and report of bearing values, etc., were made and completed prior to and separate from the development of the detailed structural designs. For the subject project, an initial phase of the foundation exploration was undertaken as an aid to site selection. This provided some data for preliminary design studies. Then, as the structural requirements and possible foundation types received detailed consideration, supplementary explorations were made of the soil structure at the particular locations of important units. The data obtained from laboratory tests of these soils were employed in studies of the capacities, settlements and costs of alternate foundation schemes being considered to fulfill the structural requirements. This process frequently involved several preliminary consultations between the structural engineers and the foundation engineers, and approximate design studies were often made before the final design was selected for complete analysis.

It has often been stated that differential settlements are the ones which usually cause trouble in engineered structures. However, the construction of an entire project or even a complete single unit of a project cannot be carried out so that the foundation load is applied at a uniform rate over the entire area of the unit. Therefore, the time-rates of settlements as well as the total settlements often have an important bearing upon the differential movements between portions of the plant, and upon conditions of strain or stress which may be created by these movements. For the subject project, the time-settlement studies presented by the Authors (at the lower right corner of Figure 7) were prepared to enable consideration of the effects upon the structure which might be caused by settlement during the dewatering period, and during the periods of applying structural loads. For these studies, the stress conditions in the soils were evaluated by means of Newmark's[2] tables of stress distribution. The studies were based on consolidation test data obtained in the laboratory using 2-1/2 inch diameter by 1 inch thick core samples of the soils retained in thin brass sleeves. The sleeves surrounding the

1. Senior Resident Partner, Dames & Moore, San Francisco, Calif.
2. Reference, Newmark "Influence Charts for Computation of Stresses in Elastic Foundations" Engineering Experiment Station Bulletin, Series No. 338, University of Illinois, November 10, 1942.

samples enabled some of the friable sandy loam and weak clayey soils to be handled and tested with a minimum of sample disturbance.

The settlement data presented by the Authors is of particular importance, since it gives detailed information from the start of the foundation loading period. In the writer's experience, it has often proven difficult or impossible to maintain reliable settlement records during the construction period. Such data are of major value if valid comparisons are to be made between settlement analyses based upon laboratory tests of the soils and the actual field behavior. Too often, the only settlement data available is taken after the construction period, and the results may indicate very slight settlements which are sometimes assumed to show that the computed settlements were far too large. The profession certainly has much to learn about analysis and prediction of soil behavior, and data of the nature presented by the Authors is invaluable to progress in this direction.

GEORGE L. JORDY.[1]—The writer has read the paper with great interest, even though a good bit of it is out of his specialized field, and has been extremely impressed with the thoroughness of each step in the selection of the various equipment, and in particular, the determination of the proper type of foundation, considering the varying strata and subsoil encountered and in the proper loading of the foundation mat to secure as uniform and minimum settlement as possible.

The writer's primary interest, however, in the position of Manager of the Industrial Chimney Division of The Rust Engineering Company of Pittsburgh, Pennsylvania, is the 450' by 14'9" internal top diameter reinforced concrete chimney, and he takes pride in the fact that his organization had the opportunity to collaborate in the design and construction of this unique structure. In view of this interest, naturally the writer's comments will dwell only with the chimney.

This chimney is one of the newer type chimneys handling high outflow velocity gases designed on the basis of various empirical equations as to the additional stack height that might be obtained by emitting the gases at such high velocity as to nullify, insofar as humanly possible, the combined effect of humidity, temperature and wind velocity. Naturally, the effect of these three agents can usually be achieved before the plume is bent by one or all of these agents, and this estimated height of plume is what is termed "added stack height."

By utilizing this plume effect, the actual structural height of a chimney is somewhat reduced and this, in turn, reflects on the initial economy of the structure both above grade and in foundation and sub-foundation requirements.

The Pacific Coast area is one that must be very carefully scrutinized in view of the earthquake potentiality when it comes to these higher structures, and the thoroughness with which the chimney has been investigated, both for tentative design which materially established certain clearances, as well as the engineering analysis in the final design selected, is definitely a credit to the writers of the paper.

For over three decades, I have been intimately connected with the development and growth of the chimney industry and have marvelled how some of the chimney resist conditions beyond the scope of their design. On the Pacific

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Coast there are chimneys that have been built—true, they are not as tall as this Morro Bay chimney nor as large in diameter, in many cases—that have been designed to resist nothing more than a 20# per square foot of projected area wind load, yet many of these old-time chimneys have gone through some fairly heavy earthquake disturbances. The writer's organization has had the privilege of designing chimneys for erection all over the West Coast, as well as in some of the highly dangerous earthquake areas in other parts of the universe, and it is surprising to know and realize how engineers have been floundering for many years trying to establish the right kind of formula so as to provide a safe structure under the severe earthquake conditions. Most of them create a very difficult design problem, and it is because of this that our societies, such as the American Society of Civil Engineers, the Structural Engineers Association of Northern California, the American Concrete Institute and the American Standards Society, have been striving to achieve a uniform solution. Their activities have somewhat been emphasized in the method of investigation of this Morro Bay chimney structure.

The writer does not believe that the proper design for a chimney is the application of a uniform earthquake coefficient to the entire structure any more than he believes that a uniform wind load over the entire structure is the true theoretical solution for a proper design. We, in an industry, must accept the theoretical formulas that have been established, but at the same time, we have to search for means whereby the overall economy of the structure from a design standpoint will guarantee the safety desired through the theoretical formula. The graphs shown in Figure 9 of the paper indicate the comparison of certain codes. In principle, they embody the American Society of Civil Engineers Code using the seismic coefficient of .13 and the American Concrete Institute old code where a uniform seismic coefficient of .20 was generally used in areas of greatest seismic probability. Comparing the curves of the final design with those of the tentative design, and in particular with No. 3 curve which is predicated on the American Concrete Institute 1953 Code using a seismic coefficient of .20 for the lower one-fifth of the height and an increasing coefficient for the remainder of the height, it will be noted that this latter code—the 1953 A.C.I. Code—will provide an excessive amount of rigidity, particularly in the central three-fifths of the unit.

The writer has discussed this paper with Mr. K. B. Foster, his Chief Chimney Engineer, who has been a member of the American Concrete Institute Committee 505 since its inception, and it is both his and the writer's opinion that the chimney in question could only have been better from a structural design point of view by either of the following.

1. Using a seismic coefficient of .20 for the bottom one-fifth of the chimney and for the upper four-fifths a seismic coefficient of .13 or .15, and increasing either of these coefficients in the manner prescribed by the American Concrete Institute 1953 Code, this would have provided a somewhat more rigid structure in the middle three-fifths without increasing materially the requirements of the design as built.
2. For a chimney slightly less rigid, the solution would have been to use the seismic coefficient of .13 for the bottom 90' and increase it according to the American Concrete Institute 1953 Code for the remainder of the height.

This comment for somewhat betterment in design is no reflection on anyone connected with the project. It is merely an expression of opinion, since

we do believe that additional rigidity is needed in the center three-fifths of any chimney structure subject to earthquake forces. In the Pacific Coast area, the writer believes the most pertinent illustration of this requirement might be the earthquake effect on the 375' chimney at the Los Angeles Gas and Electric Corporation, Seal Beach Power Station. This is one of the older type designed units, and we cannot entirely blame the chimney since it was a chimney that was superimposed on a structure, but because the upper portion of this chimney was substantially affected, the writer's primary indication, as expressed in the above opinion, is that consideration must be given to this area of the structures.

Under wind conditions, it has become definitely recognized within the past three years, that a varying force must be used as the height increases and this, of course demands consideration of reinforcing against the complicated stresses that occur under extreme gusty conditions in the upper areas of such structures.

The writer wishes to again compliment Messrs. Thon and Coltrin for the excellent analysis they have made in connection with this Morro Bay chimney, and it is the writer's opinion that a chimney structure has been provided that will give many years of excellent service. Only the best has been used in all parts of the unit to resist any possible defects due to erosion, corrosion and velocity pressure that might develop in these high velocity units.

Discussion of
"PRINCIPLES OF FEDERAL HYDRO-ELECTRIC POWER DEVELOPMENT"

by William Whipple, Jr.
(Proc. Paper 739)

ROSS N. BRUDENELL,[1] A.M. ASCE.—Colonel Whipple's interesting paper should be valuable in helping to consider objectively the engineering and economic principles of Federal water-control projects involving power generation. Amidst the claims and counterclaims that have been made, often for propaganda effect, his well presented discourse treats the subject in a considered manner worthy of its importance.

As the author noted, he deals largely with Federal practice as represented by the Corps of Engineers. The Bureau of Reclamation and the Tennessee Valley Authority have also developed large hydro projects and the processes and history are naturally not exactly parallel.

While the writer cannot here do similar justice to the TVA program, he would like to direct attention to the principles of Federal hydro-electric power development that are expressed in the TVA Act. The application of those principles in the Tennessee Valley region has emphasized:

1. The inherent economy of sound multiple-purpose projects developed as a part of the over-all development of the resources of a region.
2. The elastic nature of electricity use when encouraged by a promotional-rate, high-use policy.
3. Partnership with local interests through the local distribution organizations which deliver the power to ultimate consumers.
4. While giving primary attention to low rates and regional and national benefits, the Government, with an integrated system, can earn on its power investment, including allocation of joint costs made by a generally accepted method, sufficient income to cover all operating costs, depreciation, and the interest costs of Federal borrowings, to make in-lieu tax payments to local governments, and to provide additional earnings that compare rather favorably with the income taxes of utility companies.

In discussing origins of Federal power programs, Colonel Whipple describes Muscle Shoals as being a kind of historical anachronism and not a forerunner of the present Federal power program. Muscle Shoals was an early example of the proposition that the Government may well investigate and invest in the development of regional resources for the greatest good to the greatest number. Its design and construction to provide maximum benefits for both navigation and power was perhaps a forerunner of later multi-purpose Federal developments, somewhat in contradistinction to the early practice of adding a privately owned powerhouse incrementally to a navigation dam.

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Colonel Whipple's history brings out the importance of practical data on water resources as the foundamental basis for sound planning. The Corps of Engineers' "308" reports were one of the best investments by the United States in connection with later multiple-purpose developments and helped immeasurably in giving a sound foundation to many subsequent Federal developments including TVA.

The author does well to emphasize that over the nation and over the broad span of years, hydro power is essentially a valuable auxiliary to thermal sources of power upon which the ever-growing energy needs of the country depend. The relative cost of public financing as compared to private financing has been of considerable importance in the rate of Federal hydro-electric development. The low cost of Federal money has helped to make possible dam-and-reservoir projects that have conserved fuel resources and that have created multiple benefits. Another important factor has been the fact that price of labor and materials at the time of construction largely determines the costs of hydro power as compared to the changing operation and fuel costs of thermal projects. Many of the largest Federal dams have proved to be especially well worthwhile because price levels have gone up since they were built.

Colonel Whipple has succintly brought out the lengthy process involved in bringing the Corps of Engineers' projects from early planning to completed construction and the fact that inter-agency cooperation could result in overwhelming complexity if some means is not found for simplifying working arrangements. Unless there is a single responsibility for completing recommendations and for their fulfillment when authorized, it has proved most difficult to arrive at a practical answer without turning "comprehensive" into "compromise."

Colonel Whipple has dispassionately and ably analyzed controversial aspects of the economics of Federal projects. It is important to distinguish the difference, as the author has done, between maximum net benefits and maximum benefit-cost ratio or maximum profit ratio. TVA's planning for power has always been on the basis of maximizing total net benefit. In discussing value of power, it may well be emphasized that the use of sale price for determining value cannot be used in monopolistic or noncompetitive situations, as his examples illustrate. The utility industry is a natural monopoly. Many differences of opinion stem from failure to recognize this fact. Others stem from the desire to maintain the benefits of this monopoly. The use of thermal alternatives provides the best measure of value in most cases. However, the author states that "In most parts of the country the value of needed electric power must be as much as the cost of producing equivalent power from steam plants of modern design through private financing, since if the hydro plant is not adopted the thermal alternative will actually be built." It seems to the writer the proper measure for appraising the comparative value of a Federally financed hydro plant is what it would cost to produce equivalent power from steam plants of modern design with Government financing rather than private financing. Other things being equal it is doubtful that there is economic justification for the Government to construct a hydro plant if the power is more costly than if it were produced in a similarly financed steam plant. Otherwise as the author points out in another connection, Government hydro plants might be built that would utilize more materials and labor in proportion to the power produced than the steam plants they would replace.

It would appear that the introduction of "taxes foregone" into computations

of economic justification of Federal dams is partly due to the commendable effort to find a better basis for comparison between a Federal hydro plant and an alternative steam plant. However, the writer feels that introducing the speculative cost element of "taxes foregone" clouds rather than clarifies the economic choice, and is not as sound or simple an approach as comparing Federal hydro with a similarly financed steam plant. The subject of taxes is probably the most difficult upon which to obtain objective thinking instead of "resentment." Much of the "resentment" referred to, and similar objections to Federal power development arise from the idea that tax revenues, principally income taxes, are the source of capital funds for these developments. Although Treasury accounting makes no distinction, the history of the Federal deficit suggests rather clearly that Federal investments such as water control projects have in fact been financed from Treasury borrowings. In any event, if interest costs are to be computed on investments in Federal dams, the assumption is that the money was borrowed by the Treasury and not raised through taxation.

The effect of various forms of taxation and the effect of alternative economic enterprises on total tax collections are among the most complex of economic problems. Our tax system is in no wise perfect, but in the form that income taxation has taken in this democratic country, it is doubtful that there is cause for complaint if some of the revenues from a Federal enterprise are not earmarked as Federal tax-equivalents, as long as 100 percent of the net income from such a Federal revenue-producing investment belongs to the Federal Government. If the rates from a Government project were only sufficient to cover interest and depreciation as fixed costs, there would be no direct collection of tax-equivalents from the consumer through his electric bill; however, with the profitable and elastic nature of electricity use, the lower rates in turn generate new economic factors, ad infinitum, so that other tax collections from the consumer, and from those with whom he spends his savings, may increase substantially. Certainly the determination of "taxes foregone" is a complex subject; moreover, the writer doubts the practicability of determining the taxes foregone by Federal construction, except in the case of ad valorem taxes on property taken from other uses for a Federal development. For example, if a power company buys the power from a Federal dam at a price equal to or lower than its own cost of making the power, it realizes as much profit on the ultimate distribution and sale of this power and hence pays as much or more income tax as it would if it had built the plant itself; thus in such a case, no Federal taxes would be foregone.

Colonel Whipple is quite correct that the proper combination of various purposes with power must be determined by careful quantitative analysis which is highly specific to the particular circumstances. In the Tennessee Valley area, eighty years of rainfall and runoff records and previous historical information have shown a distinct and predictable reduction in flood flows and volumes after the normal flood season of winter and spring. This fact has been used to permit considerable combined-purpose storage in TVA reservoirs—storage which is evacuated on a seasonal schedule, with flood control requirements taking priority over power needs. Larger evacuations of storage which may be made for power purposes supply incidental flood storage benefits, and add to the flood control benefits rather than encroaching upon them.

The author was probably not intending to imply (pages 8-9) that in the East flood control and power production are largely conflicting interests in

multiple-purpose hydro development. It is not safe to generalize on this subject. In the Tennessee Valley TVA has obtained both jointly at less cost than both could be obtained separately to the same degree.

The author has done a service in pointing out some of the relationships of thermal power to hydro power that have not always been fully understood. Too much emphasis has often been given to the firm continuous output of each hydro project. The greatest value for hydro power is frequently obtained by installing a large amount of capacity. Nevertheless this should not be taken to imply that it is always economical to maintain all the capacity as "dependable" capacity regardless of system conditions. The statements that may serve as a broad generalization may not accord with the facts in specific instances. For example, hydro capacity, even with a minimum of energy supporting, may often be more useful than an equal amount of steam capacity because of the ease and speed with which it can pick up load; in other cases it may have substantially less usefulness than equal steam capacity because in emergency the steam plant is able to back up its capacity with an energy supply which is available practically 100 percent of the time.

The advisability of not "selling short" the peaking capabilities of hydro sites in making provision for ultimate installations has been borne out in practice in the Tennessee Valley region. All the vacant stalls in the TVA hydro projects will be filled by next year despite numerous early predictions that such capacity could never be used.

As Colonel Whipple suggests, operating criteria should be developed to carry out results which a project is intended to achieve. However, changing system conditions may necessitate year-to-year differences in the operations if the optimum results are to be obtained. Conditions of both power supply and power requirements vary. Moreover, it is by capitalizing on additional incremental values that could not be foreseen in the initial planning that the economic benefits of a project may be brought to full fruition.

The writer would like to point out, in connection with discussions of cost allocations of Federal multiple-purpose projects, that the "separable cost-remaining benefits" method upon which the Corps of Engineers, the FPC and the Department of the Interior agreed last year is essentially the same as TVA has used since 1938 under the name of the "alternative justifiable expenditure" method (H. D. 709, 75th Congress, 3d session).

Discussion of
"THEORY FOR THE DESIGN OF UNDERGROUND PRESSURE CONDUITS"

by D. J. Bleifuss
(Proc. Paper 741)

EVAN W. VAUGHAN,[1] M. ASCE.—The author's brief treatment of a very interesting and complex design problem introduces two original ideas worthy of more detailed study and re-states an old and accepted theory.

The concept of a cylinder of cracked rock within a cylinder of intact rock, the corresponding deformation equations, and the equating of the deformation of the rock to that of the steel have been used for a sufficient length of time to be considered as the standard theoretical approach. The author's equation (8) will be recognized as Lame's equation. Its approximate form, for extremely thick-walled cylinders and its application to steel-lined tunnels was presented by C. P. Dunn,(1) M. ASCE, over 30 years ago. In a discussion of that paper, B. F. Jakobsen, introduced the idea of an inner cylinder of cracked rock and the pertinent deformation equation (author's equation (10)). Since that time a number of papers have been published on the theory of steel lining design, among which might be mentioned the recent works of Trub,(2) Kastner,(3) and Hutter and Sulser.(4) Consequently, it is difficult to understand the author's statement in Conclusion that, to his knowledge, "no one has previously made any theoretical approach at all to the determination of the proportioning of load between steel and rock."

What is required to clarify the situation is not repetition of a theory that has achieved almost universal acceptance but rather an analysis of the probable limits of applicability of the theory, simplification of calculations and realistic suggestions for improvement.

The chief defects of the theoretical approach are: (a) the impossibility of determining the outer limits of the cylinder of cracked rock and, (b) the difficulty of determining acceptable values for the elastic constants of the rock in place.

In order to define the limits of the cracked rock zone, the author suggests application of the elastic theory (author's equation (4)). This is an original suggestion and at first glance appears to have considerable merit. Further investigation proves otherwise. The equation requires a value for Poisson's ratio and the proper value to use can not be determined for the rock in place, even by a field loading test. The chances are that the ratio varies both with depth below ground surface and with orientation of the major stress.

The author uses a value of 0.2 in his illustrative example and finds that the lateral compression due to the weight of the overlying rock must equal 0.287h in order to satisfy his condition that the cracked rock zone will extend outward until it reaches a point at which the compression due to gravity forces is equal to the tension resulting from the internal water pressure in the shaft. This results in a cylinder of cracked rock having an outer radius

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equal to 3.6 times the inner radius. Had he chosen a different value for Poisson's ratio the thickness of the cracked rock cylinder would have been quite different. For example, this ratio could have been taken as 0.15 or 0.30, both values being as reasonable as the one selected. For these two values, the lateral stress would be 0.203h and 0.491h, respectively and the corresponding ratios of diameters would be 5.06 and 2.09, respectively. Thus, the thickness of the cylinder of cracked rock can vary within wide limits, depending upon the value of Poisson's ratio selected.

The considerable variation indicated in the proportions of the cylinder is of less real importance than might be expected, as can be shown by the following simple analysis of the two cylinder equations.

Equations (8) and (10) give identical values for deformation when the ratio of external to internal radius is 4 and Poisson's ratio is assumed at 0.2. For smaller values of the radius ratio, equation (8) gives greater deformation than equation (10) while, for larger values, the opposite is true. Because of this peculiarity, the author could have used equation (8) instead of (10) for calculating $d_i$ in his illustrative example with no practical difference in his final result.

The basic reasoning which led to the original assumption of a zone of cracked rock adjacent to the bore is somewhat different than the thought behind the author's equation (4). Excavation disturbs the surrounding material and results in a redistribution of stress, deformation of the cross-section and shattering of the surrounding rock. The zone of disturbance is probably limited and it has been suggested by Jakobsen and Terzaghi(5) that it does not extend into the rock more than one tunnel diameter. Considering the natural arching action of the rock surrounding the bore and the probability that compressive forces in the zone of arching exceed the vertical pressure of the overlying rock, the writer believes that the assumption of a 3 to 1 radius ratio for the inner cylinder is adequate.

As a result of the characteristics of the cylinder equations and the probability that the effective cracked rock zone is relatively limited in extent, the writer developed the following single equation for the determination of the rock deformation to be used in preliminary studies or for developments of moderate size for which field deformation tests are not warranted.

$$d = 2.5 \frac{(P - P_s)r}{E_r} \qquad (a)$$

With a properly selected safety factor, based upon an evaluation of conditions, particularly geological, at the site of any particular development, this equation should result in a safe design and has the advantage of simplicity. It is interesting to note that when applied to the author's example, it gives a value for d of 0.0193" as compared to his own value of 0.01916".

In regard to the elastic constants, the author suggests that the effective modulus of elasticity be determined by a test of the rock in place. To this suggestion practically everyone who has ever worked with the problem will agree. However, it is not necessary to find the elastic modulus as a separate factor nor is it necessary to retain in detail the concept and complex equations utilized in the purely theoretical treatment.

The equations for the rock cylinders show that the deformation is proportional to the internal pressure times the radius. Therefore, if the test is

made in an adit, geometrically similar to the prototype conduit and sufficiently long to reduce end effects to negligible values and if the adit is driven in rock of similar character and similarly situated with respect to the rock surface, the test will be a model test and the proportionality indicated above can be applied directly. Thus one simple slide rule operation is all that is needed to determine the deformation for the prototype from the test observations and no further consideration need be given to the specific elastic constants and the relative dimensions of the cracked rock cylinder. Since the relation between load and deformation is not rectilinear, the pressure used in the test chamber should be sufficiently great to allow use of the test curve directly without extrapolation. A number of tests of this type have been made(6) and where the rock formation is massive and watertight, no lining of any kind is required in the test chamber. This type of test has several advantages: deformation measurements are made directly on the rock, leakage through the rock can be determined, and the effects of repetitive and sustained loading can be measured.

The author's method of test appears to be both expensive and complicated. Since a steel sphere is used, no geometric relationship exists and a complex calculation is required to deduce the rock modulus from deformation measurements of the steel. It is necessary also to assume that intimate contact exists between the steel, concrete and rock or to locate from the test curve the load at which the rock first begins to deform. It is probable that the initial increments of load will be taken entirely by the steel.

It is possible to make the deformation test on a steel lined section of the test adit and such a lining should be used if the rock is not reasonably watertight. In this case the test will be a model test of the completely lined conduit. The end effects can be reduced to negligible proportions if the test section is made long enough and this should be no more expensive than the author's elaborate test, since a fairly long steel cylinder and test head would cost no more to fabricate than the steel sphere which he suggests using.

In applying the deformation relationships, the author's equation (12) makes no allowance for thermal contraction of the steel after completion of construction nor does it take into account the possibility of plastic yielding of the rock under sustained loading. These factors should not be neglected.

Despite the recognized weaknesses of existing methods for proportioning the steel lining to resist internal pressure, these methods, used intelligently, have produced a great number of successful conduits. Failures due to bursting pressure are extremely rare. On the other hand, buckling failures, due to external pressure, have been relatively common. The author gives no solution to this important problem.

The writer's critical remarks notwithstanding, the author deserves congratulations for his courage in presenting his views of an interesting and still controversial design problem.

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STAVROS N. NICOLAOU,[1] A.M. ASCE.—Mr. Bleifuss has presented a paper on one of the main problems of underground power stations, namely, the design of underground pressure conduits.

Steel lined shafts have been designed in the past to withstand the full internal hydrostatic and dynamic pressures, with allowable working for the steel stress equal to or less than the yield stress. However, this assumption did not sufficiently take into account the rock mass surrounding the steel liner. Rock in great depth behaves as a homogeneous material, as Mr. Bleifuss has assumed, with the rock pressure approximately the same in any direction at any particular point. Field measurements inside tunnels and galleries have shown that this assumption is approximately correct.(1)

The analysis by Mr. Bleifuss assumes the partial transmission of the water pressure from the steel liner to a surrounding "cracked" zone of rock, and to an outer zone of homogeneous rock. A similar assumption was made for the design of the underground penstocks at the Kemano project in Canada.(2) The author's calculations, however, do not include the effect on the stresses in the steel liner resulting from:

a) The difference in temperature between the surrounding rock and the water during initial operation.(3)

b) The concrete lining between steel liner and rock.

An analysis will be made taking into account the temperature differential and the concrete lining around the steel liner. For this analysis, the following notations will be used (see Figure 1), in addition to those used by Mr. Bleifuss:

$P$ = internal hydrostatic pressure, in p.s.i.
$P_t$ = internal pressure required to eliminate the gap $d_t$, in p.s.i.
$P_\eta$ = $P-P_t$ = internal hydrostatic pressure applied to steel liner, in p.s.i.
$P_c$ = unit radial pressure in concrete lining at a radial distance "c," in p.s.i.
$d_t$ = gap between steel liner and concrete lining due to the temperature difference, in inches.
$d_c$ = radial deflection of concrete lining at radial distance "c," in inches.
$f_t$ = unit circumferential stress in steel liner due to the temperature difference, in p.s.i.
$f_s$ = unit circumferential stress in steel liner due to internal hydrostatic pressure, in p.s.i.

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$f = f_t + f_s$ = total unit circumferential stress in steel liner, in p.s.i.
$f_{all}$ = allowable circumferential working stress in steel liner, in p.s.i.
$\mu = 0.20$, Poisson's ratio for rock.
$\epsilon_s = 0.0000065$, coefficient of thermal expansion of steel, °F.
$\epsilon_r = 0.0000036$, coefficient of thermal expansion of granite, °F.
$E_c$ = 3,000,000 modulus of elasticity of concrete, in p.s.i.

## Analysis

### a) Conditions due to Temperature

It is very difficult to predict underground temperatures in tunnels and shafts. The study of such temperature variation is based only on experience. The geothermic gradient, i.e., the depth below ground surface corresponding to an increase in temperature of 1°F, depends on the thermal conductivity of the rock. The thermal conductivity of the rock is a function of the types of constituent rock minerals, chemical changes taking place, moisture content, structural features including altitude of bedding, joints, faults, cleavage, etc., and other factors. For granite and crystalline schists, the geothermic gradient generally ranges between 100 ft. and 140 ft. In the Simplon Tunnel in Switzerland where the rock was comprised of gneiss, mica schists and some limestone, the average gradient was from 70 ft. to 80 ft.(4)

An average gradient of 1°F for each 60 ft. of depth is a reasonable assumption. Assuming ground surface temperature is 60°F, the rock temperature at 500 ft. is approximately 70°F. With water flowing through the pressure shaft at 50°F, the initial temperature difference $\Delta t$ between surrounding rock in Zone B and water is $\Delta t = 20°F$, and:

$$d_t = \Delta t \epsilon_s r + 0.5 \Delta t \epsilon_r r \quad (3)$$

$$d_t = 0.00016r \quad (1)$$

$$P_t = \frac{0.00016 E_s t}{(1-\mu_s^2)r} \quad (2)$$

### b) Conditions in Concrete Lining - Zone A

Assume radial cracks in the concrete lining. Radial cracks result when the hoop tension transmitted by the liner exceeds the tensile strength of the concrete. The concrete liner serves only to transmit the pressure from the liner to the rock. The equations of equilibrium and deflection, respectively (see the writer's Fig. 1) are as follows:

$$P_c c d\phi = (P_n - P_s) r d\phi \quad (3)$$

$$d_c = \frac{P_c (c-r)}{E_c} \cong (P_n - P_s) \frac{c^2 - r^2}{2E_c c} \quad (4)$$

c) Conditions in Rock - Zones B and C

For zones B and C representing cracked and homogeneous rock respectively, the relationships derived by Mr. Bleifuss are applied, but also taking into account the temperature and the concrete lining conditions.

General Equations

Equating the deflections of the steel liner and the surrounding materials in Zones A, B and C:

$$d_s = d_c + d_i + d_o \qquad (5)$$

Substituting in (5) the writer's value for $d_c$ from (4) and Mr. Bleifuss' values for $d_s$, $d_o$ and $d_i$ relationships (5a), (8) and (9) respectively of the author's papers, and expressing "a" in inches:

$$\frac{P_s r^2(1-\mu_s^2)}{E_s t} = (P_n - P_s)\left[\frac{c^2-r^2}{2E_c c} + \frac{r}{E_r}\log_e \frac{a}{c}\right] + \frac{P_s a\,(1-\mu)}{E_r} \qquad (6)$$

Also:
$$P_n = P - P_t = P - \frac{0.00016 E_s t}{(1-\mu_s^2)\,r} \qquad (7)$$

Application

For a comparison with Mr. Bleifuss' results, the same data are used for a 75 inch diameter steel penstock, with a 12 inch concrete lining. The final circumferential stresses of the steel liner are derived by solving relationships (6) and (7) by trial and error, and are tabulated in the writer's Table I. The stresses in the steel liner, neglecting the temperature effect, are very close to those computed by Mr. Bleifuss.

The writer's Fig. 2 is a plot of the dimensionless parameter $\frac{P_s}{P}$, $\frac{P'_s}{P}$ and $\frac{f}{f_{all}}$ versus $\frac{t}{r}$. This graph shows the effect of the temperature difference on the stresses in the steel liner. For a thickness of steel of 1 inch, temperature stresses amount to 25% to 30% of the total circumferential stresses of the steel liner. It is also apparent that the concrete lining around the liner has very little effect on the stresses transferred to the rock.

As an illustrative example, the grade of steel used is Carbon Steel ASTM A-285 Firebox C, with yield stress equal to 30,000 p.s.i. Curve 1 is based on the formula for thin cylinders with full internal pressure taken by the steel liner up to the yield stress. Curves 2 and 3, respectively, correspond to data from the writer's Table I and the author's Table I wherein the allowable working stresses are equal to 2/3 of the yield stress. It is interesting to note that considerably different values for steel thickness are obtained, depending upon the assumptions made. For $f = f_{all}$, and $r = 37.5"$.

Curve 1, $t = 0.90" \cong 1.0"$, and $P_s = P$
Curve 2, $t = 0.56" \cong 0.6"$, and $P'_s = 0.4P$, where $P'_s = P_s + P_t$
Curve 3, $t = 0.27" \cong 0.3"$, and $P_s = 0.2P$

Another important effect on the stresses taken by the steel liner which has not been included in the above analysis is the gap or void existing between the steel liner and the concrete, and between the concrete and the rock. This gap occurs from shrinkage due to the setting of the concrete and results from high temperature rise and subsequent contraction and temperature drop. The effect of the gap or voids on the stresses in the steel liner can be offset by an extensive program of low pressure grouting. This program should consist of two stages. In the first stage, grout is injected at low pressures between the concrete lining and the rock. In the second stage, which should take place fifteen to thirty days later, grout is injected at low pressures between the steel liner and the concrete. Grout pressures from 70 p.s.i. to 100 p.s.i. have been used for both stages in numerous steel lined pressure tunnels or shafts.(5)

In cases where such low pressure stage grouting is not extensive or where the grouting technique has not obtained the desired results, the stresses due to the gaps or voids may amount to as much as 100% to 150% of the temperature stresses. In these cases, the combined stresses from internal pressure, due to both temperature differences and gaps between steel liner and concrete lining, may amount to as much as 50% to 70% of the total stresses taken by the steel liner.

Additional grouting of the rock at higher pressures 150 p.s.i. to 400 p.s.i. aids in the redistribution of the stresses taken by the rock by prestressing the rock; such grouting also induces compression of the concrete lining and the steel shell. With efficient and extensive consolidation and cut-off grouting in fissured rock, or in the cracked zone of rock around the liner, the portion of the internal pressure transmitted to the steel liner can be further reduced.

Concrete which has little or no setting shrinkage, is at present the best assurance of monolithic encasement. Instead of using conventional concrete, "prepakt" concrete may be used, as was the case at the Kemano project. The point to point contact of the aggregate pieces and the rock together with the use of cement base intrusion mixture minimizes the contraction of the concrete. Prepakt concrete 2 ft. in thickness was placed at the Kemano project, employing a process of low pressure grouting.(6)

Simplification of Trial and Error Method Presented by Mr. Bleifuss

To avoid the trial and error method used by Mr. Bleifuss to determine the pressure Ps transmitted to the steel liner, the following relationships are derived:

From the writer's relationship,(5) neglecting the temperature effect and the concrete lining, and solving for Ps:

$$P_s = \frac{P}{1 + \dfrac{1 - \mu_s^2}{\frac{t}{r} \cdot \frac{E_s}{E_r}\left(\log_e \frac{a}{r} + 1 + \mu\right)}} \qquad (8)$$

Expanding $\log_e \frac{a}{r}$ into a series:

$$\log_e \frac{a}{r} = 2\cdot\left[\frac{a-r}{a+r} + \frac{1}{3}\left(\frac{a-r}{a+r}\right)^3 + \frac{1}{5}\left(\frac{a-r}{a+r}\right)^5 + \cdots\right] \simeq \frac{2a-2r}{a+r} \qquad (9)$$

From the author's relationship (7), where "a" is expressed in inches:

$$\log_e \frac{a}{r} \simeq \frac{2a-2r}{a+r} = \frac{2(P-P_s-P_a)}{P-P_s+P_a} \qquad (10)$$

$$\left(3+\mu+\frac{1-\mu_s^2}{\frac{t}{r}\cdot\frac{E_s}{E_r}}\right)\cdot P_s^2 - \left[6P+2P\mu - P_a+P_a\mu + \frac{1-\mu_s^2}{\frac{t}{r}\cdot\frac{E_s}{E_r}}(P+P_a)\right]P_s +$$

$$+ P^2(3+\mu) - P\,P_a\,(1-\mu) = 0 \qquad (11)$$

Relationship (11) is a quadratic equation with Ps as the unknown and offers a direct solution for Ps. Using the author's assumed data as an illustrative example, and using the writer's relationship (11) the steel liner stresses are tabulated in the writer's Table II. The stresses are practically identical to those computed by the trial and error method of Mr. Bleifuss.

## CONCLUSIONS

From this discussion, it is apparent that the temperature differences and the shrinkage effect of the concrete encasement should be included in the analysis to determine the percentage of the internal pressure transmitted to the liner. However, Mr. Bleifuss' analysis supplemented with the aforementioned effects can be used only as an approximate method, to determine the stresses in the steel liner. Field measurements and methods to determine rock properties such as modulus of elasticity, crushing and tensile strength, and permeability are difficult, and do not always offer adequate data for use in the theoretical analysis. The effect of the modulus of elasticity of the rock in the determination of the portion of the internal pressure taken by the steel liner has been shown in the author's Table 1. The analysis should be based whenever possible on the value of modulus of elasticity for the rock as determined in the field after the rock has been grouted.

Another important factor in the design of underground pressure conduits, mentioned by Mr. Bleifuss, is the external hydrostatic pressure acting on the pressure conduit when empty. This condition occurs infrequently, but sometimes governs the design of steel liners, especially when the pressure conduits are situated very close to a reservoir. In this case, stiffening rings or anchorages into the rock may be used to stiffen the steel shell to insure against buckling. Some designers, however, avoid the use of stiffeners and anchorages because of the difficulties encountered in placing grout and concrete. Where stiffeners are not used one solution is to design the steel liner with an adequate plate thickness to withstand an external hydrostatic pressure equivalent to the vertical distance from the center line of the conduit to the surface of the ground.

The problem of the design of steel liners for underground pressure

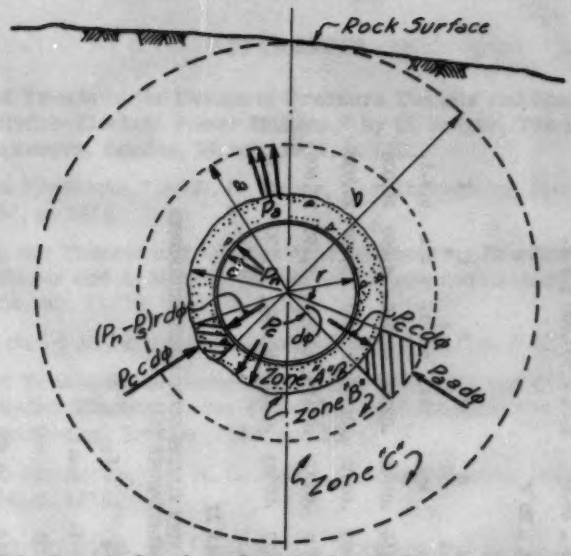


FIG. 1 Cross Section Steel Lined Pressure Conduit

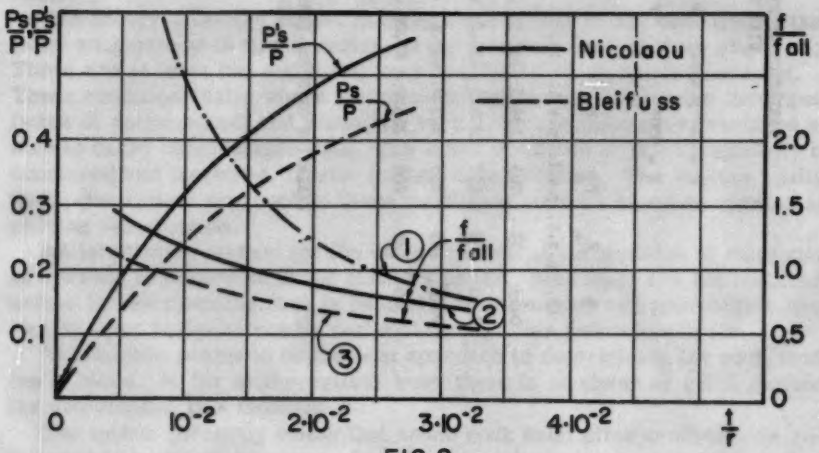


FIG. 2

TABLE I

VARIATION OF STEEL LINER STRESSES IN PRESSURE CONDUITS
WITH VARYING STEEL LINER THICKNESSES

$P = 740$ p.s.i. $b = 500'$ $c = 49.5"$ $E_T = 2{,}500{,}000$ p.s.i.

$h = 500'$ $r = 37.5"$ $t = 20°F$

| $t$ | $P_t$ | $P_n$ | $P_s$ | $a$ | $d_c$ | $d_i$ | $d_o$ | $d_s$ | $f_s$ | $f_t$ | $f$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| in. | p.s.i. | p.s.i. | p.s.i. | in. | in. | in. | in. | in. | p.s.i. | p.s.i. | p.s.i. |
| 0.25 | 35 | 705 | 138.5 | 147.5 | .00199 | .01165 | .01020 | .02384 | 20,750 | 5,250 | 26,000 |
| 0.50 | 70 | 670 | 209.0 | 120.2 | .00162 | .00803 | .00831 | .01796 | 15,680 | 5,250 | 20,930 |
| 0.75 | 105 | 635 | 247.0 | 101.1 | .00136 | .00579 | .00701 | .01416 | 12,350 | 5,250 | 17,600 |
| 1.00 | 140 | 600 | 266.0 | 87.00 | .00118 | .00423 | .00603 | .01144 | 10,000 | 5,250 | 15,250 |

TABLE II

VARIATION OF STEEL LINER STRESSES WITH VARYING STEEL LINER THICKNESSES
DERIVED FROM RELATIONSHIP (11)

| $t$ | $P_s$ | $f_s$ |
|---|---|---|
| in. | p.s.i. | p.s.i. |
| 0.25 | 131 | 19,630 |
| 0.50 | 217 | 16,300 |
| 0.75 | 280 | 14,020 |
| 1.00 | 325 | 12,190 |

conduits, whether the external or internal water pressure conditions govern, is a matter of judgment, economics and adequate knowledge of the natural site conditions.

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3. "Beitrag zur Theorie und Konstruktion gepanzerter Druckschacte," by Von A. Hutter and A. Sulser, Wasser und Energiewirtschaft-Cours d'eau et energie, Nr. 11/12, 1947; p. 145.
4. "Precis de Geologie" by Leon Moret, Paris, 1947, pp. 5-6.
5. "Present Trends in the Design of Pressure Tunnels and Shafts for Underground Hydro-Electric Power Stations," by C. Jaeger; The Institution of Civil Engineering, London, 1955, p. 150.
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WM. REID MARTIN, JR.,[1] and JAN A. VELTROP,[2] Junior Members, ASCE.—The paper presented by the author treats a major problem of modern hydroelectric project design. This is a welcome beginning and the writers hope that a thorough understanding and basic criteria may be the final outcome.

The theory proposed seems to be applicable only to one condition of the many encountered in tunnel design, namely tunnels through deep sound rock. There are at least two additional rock conditions which merit treatment. These conditions exist where the surrounding rock is completely incompetent (weak or decomposed) and where the rock is heterogeneous or fractured but able to carry considerable load. The latter condition is more frequently encountered and therefore, merits further consideration. The writers realize that a theoretical approach to these conditions must be based on similar simplifying assumptions.

An interesting method for the determination of the modulus of elasticity of the rock is presented in the author's paper. Such tests are not justified unless the surrounding rock is basically homogeneous and appreciable savings in liner material can be realized through this refined analysis.

The method seems to be the best approach to determining the rock modulus in place. So far as the writers know there is no cheap or quick method for determining this modulus.

The author correctly states that sound rock must offer considerable resistance to bursting pressure. In order to evaluate the contribution of the rock it is necessary to make assumptions regarding the condition of stresses

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2. Design Engr., Harza Eng. Co., Chicago, Ill.

existing in the rock. Without giving any explanation the author adopts a relationship between the horizontal and vertical stresses, which appears to be based on the assumption that the rock is laterally restrained. This assumption differs from the Heim theory,[3] which assumes that rock at great depths behaves in a plastic manner or even as a liquid under its own weight. The pressure at any point, according to the Heim theory, would be the same in all directions and equal to the hydrostatic pressure using the density of rock. The authors method realizes only about 0.2 of the hydrostatic value in the lateral directions. Other investigators have found that the magnitude of stresses in the rock is much higher than can be accounted for by the weight of overlaying strata.

Another major assumption made concerns the interaction between the steel liner and the rock. Here the author postulates that the radial deflections of the liner must equal the radial deflections of the rock. This implies initial contact between the steel liner and the concrete shell, which the author apparently accepts without question, and without reference to the difficulties involved in attaining such an ideal situation. Any gap existing between the steel and the concrete (eg. due to temperature differences between the water and the rock) unless some degree of initial prestress is created by grouting, has to be closed by stretching of the steel thereby creating tensile stresses in the liner which increase with the size of the gap.

The assumptions concerning the ration of b/a, limit the applicability of the author's theory to conduits at great depth. The effective outer radius (b) of the thick cylinder can only equal the height of rock above the centerline of the tunnel if the rock is sound for this full depth. This limitation excludes application to design of many conduits whose depth to diameter ratio is much smaller than the example of the author.

The forming of a "cracked" zone around the liner in sound rock seems very questionable. Other theories have been proposed regarding what happens if radial fissures were to develop in the rock. These, as well as the author's theory, are not convincing since in numerous installations tensile stresses which develop in the rock appear to have no effect on the safety of the liner.

The many uncertainties inherent in the problem, make a scientifically exact solution impossible. The author states that his theory should at least give an indication of the order of magnitude of the stresses in the steel lining. It would be of interest to designers to know what the author considers to be an acceptable stress which would assure safety consistent with economy. In the example given no criterion is suggested for comparison with computed stresses.

The problem of external pressures which often govern the design of steel liners is mentioned. The author merely observes that the liner should not buckle. Mr. Jaeger points out that "Most designers agree that external water pressure is a major problem, but opinions diverge considerably regarding the distribution of the external pressure and even more so regarding the methods of stiffening the steel liner and the design of the liner against buckling."

Because of the many practical limitations basic to the theories which include the modulus of elasticity of rock, the radius of a cracked zone around the tunnel, and the effectiveness of the contact between the liner and the

3. A. Heim, "Tunnelbau und Gebirgsdruck" ("Tunnel work and rock pressure") Naturf. Gesell., Zurich, Vol. 50 (1905), p. 1.

concrete shell, the methods presented for design must necessarily encompass many more conditions than the one proposed by the author for tunnels in deep sound rock.

The subject of the design theory for pressure tunnels has been treated by previous authors. A most comprehensive treatise is given in a recent paper by Charles Jaeger, entitled "Present Trends in the Design of Pressure Tunnels and Shafts for Underground Hydro-Electric Power Stations."[4] Mr. Jaeger also presents an extensive bibliography on the subject.

M. L. PEI,[5] J.M. ASCE.—In order to determine the stresses naturally existing in the rock, the author stated without proof the following equations:

$$\sigma_z = \mu(\sigma_x + \sigma_y) \tag{1}$$

$$\sigma_x = \mu(\sigma_z + \sigma_y) \tag{2}$$

where Poisson's ratio, $\mu$, is universally defined as

$$\epsilon_z = -\mu(\epsilon_x + \epsilon_y) \tag{21}$$

$$\epsilon_x = -\mu(\epsilon_z + \epsilon_x) \tag{22}$$

Any standard textbook of elasticity(1) will show that eq. (1) is valid if the longitudinal displacements of every point in rock along the z-axis are zero. This is the so-called plain strain case and reduces the problem to a two-dimensional case. This is a reasonable assumption.

However, for eq. (2) to be valid, it is necessary to make an additional assumption that there is no displacement along the radial x-axis, and in fact reduces the problem to an one-dimensional case. This assumption is obviously unreasonable.

A rigorous solution of stresses in rock around a tunnel was given by Yu.(2) Terzaghi and Richart(3) also indicated the range of possible values of stresses near the tunnel. According to these writers, $\sigma_x$ is generally much greater than $\sigma_y$, rather than equal to $0.25\sigma_y$, as indicated by author's eq. (3).

The author was apparantly unaware of previous attempts to determine division of load between the liner and rock. Gumensky and Chu,(4) for instance, had solved the problem based on the same consideration of equal radial displacement.

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2. Y. Y. Yu, "Gravitational Stresses on Deep Tunnels" J. Applied Mechanics, vol. 19, No. 4, Dec. 1952.

4. Proceedings of the Institution of Civil Engineers, March 1955 (London, England).

5. Lecturer, Civ. Eng. Dept., The City College of New York, N. Y.

3. Terzaghi and Richart, "Stresses in Rock About Cavities" Geo-Technique, vol. 3, pp. 57, June 1952.

4. Gumensky and Chu, "Design of Steel Lined Concrete Pressure Conduits" Civil Engineering, November 1948.

## Discussion of "HYDRAULIC PRESSURE IN CONCRETE"

by T. C. Powers
(Proc. Paper 742)

A. HRENNIKOFF,[1] M. ASCE.—The writer is very interested in Mr. Powers' paper, especially the part dealing with the negative hydrostatic pressure in hydrated cement and concrete. This subject has always raised some doubts in the writer's mind, and he welcomes this opportunity to thresh them out in an open discussion.

When speaking of negative hydrostatic pressure in porous material brought about by capillarity, it is necessary to remember two important aspects of the situation: the presence of compression in the solid skeleton and the increase in the tension strength of the solid material. This is illustrated in figures 1(a) and (b) representing part of a porous solid body or an aggregation of a large number of discrete solid particles, the voids between which are filled with water. The voids in the rest of the body, not shown in these figures, are filled with air. As the water dries, the menisci at the boundary of water and air are drawn in, and the surface tension in them pulls the grains together. The water pressure in the voids then drops below the atmospheric: the sharper the curvature of the menisci, the greater is the drop. If a surface, such as 1-1, closely approaching a plane is now passed across the body separating it in two through the points of intergranular contacts, the resultant force on this surface must be zero, because no external forces, apart from gravity, are acting on the body. Since, however, negative pressure t is present in the water cut by the surface 1-1, there must also be some positive pressure or intergranular compression c, acting on the grains at their points of contact in order to effect the balance of forces. If the area of grain contacts on the surface 1-1 forms only a small fraction of the total area, such as one percent, the intensity of intergranular compression must be large, and in this case must become 99t. The grains so compressed at the points of contact are also subjected to the negative pressure t on the rest of their surfaces, as indicated in figure 2. A similar situation exists in vertical capillary tubes, in which the capillary rise and the presence of negative pressure in the water is accompanied by compression in the walls of the tubes themselves.

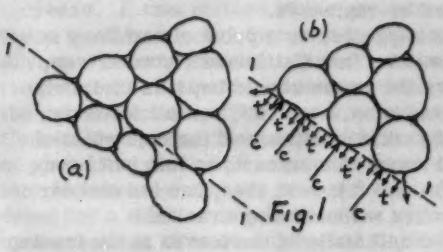

Fig. 1

Again, if the body shown in figure 1 is subjected to tension in the surface 1-1 by some external tension force, its tension strength, which in the dry

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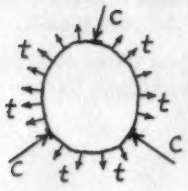

Fig. 2.

material is solely determined by intergranular bonds, is now increased by the negative pressure t in the water, because in the process of pulling the body apart it is necessary to overcome, in addition to the bonds holding the particles together, the full extent of the negative pressure in the water.

A question naturally arising in this connection is whether it is possible for the water to be actually in tension, rather than just in a state of compression smaller than the atmospheric pressure. In other words, does ordinary liquid water have an appreciable tension strength? It is, of course, clear that the everyday experience of dipping a hand into water and taking it out with practically no effort is no indication of the lack of tension strength on the part of water. It is only a proof of the lack of shearing strength. It is also realized that certain theoretical considerations of physical chemistry related to the energy associated with extension of water surface suggest the existence of high tension strength in water, but can this proposition be accepted without experimental proof? Nature often turns out to be more complicated than it appears at first glance. In this particular case, a statement that water can stand tension and, more than just that, a tension amounting to as much as 14,000 #/sq. in., or nearly 1,000 atmospheres, as suggested by the author, is something that would need strong experimental confirmation before it could be accepted by engineers.

The rise of moisture in tall trees is suggested as a proof of capillary action accompanied by actual tension in water. In a California redwood tree, 300 ft. tall, the water in the leaves near the crown should be, it is claimed, at an absolute tension of nearly 8 atmospheres, or -115 #/sq. in. However, this tension in the water would place the solid substance of the cell walls of the leaves in a compression of several times this amount, as has just been explained in connection with figures 1(a) and (b); how, then, can the slender framework of the cells stand this pressure without being crushed?

Furthermore, the compression in the cell walls of the leaves at the treetop would create entirely different conditions of growth than in leaves near the ground, and the response to the environment should produce a perceptible difference in the structure of the leaves at the top and at the bottom of the tree. However, no such difference is observed. Yet a less drastic deviation from normal conditions, when a tree grows at an angle to vertical, so that its trunk is subjected to bending, results in the formation of the so-called compression wood, which is structurally so different from normal wood as to be useless as a construction material.

Capillary rise of water plays an important part in the behaviour of soils. Some 15 - 20 years ago the writer heard statements from the authorities on Soil Mechanics to the effect that capillary rise of hundreds of feet is common in fine grained soils and that it is accompanied by appropriate very high tension in the water filling the soil voids. Strangely enough, however, most of the books on Soil Mechanics written in the last 10 years are silent on this point and some even express a different view. Thus, D. W. Taylor states that the actual tension strength of water is only one kig/sq. cm. (one atmosphere), although in fine capillaries it may be somewhat higher.[1] G. Tschebotarioff

1. "Fundamentals of Soil Mechanics" by D. W. Taylor, page 145, Wiley & Sons, 1948.

refers to capillary rise in colloidal soils of 1,200" and more, with corresponding tension in water of 43.5 #/sq. in. and over, but carefully notes that these are all theoretical figures.[2]

Consider now the cement paste. The author refers to self-desiccation of cement on hydration, a phenomenon which was also observed by the writer. The primary cause of self-desiccation is that the absolute volume of hydrated cement is somewhat smaller than the absolute volumes of unhydrated cement and water, although it is much greater than the volume of the unhydrated cement alone. Self-desiccation proceeds not only when the cement is freshly mixed but also when it is hard and in fact as long as there is any hydration in progress. In the course of self-desiccation bubbles of water vapour may very well form in the water in the voids between the cement grains. The presence of positive pressure in the vapour bubbles would not, however, preclude the possibility of high tension in the water around the bubbles, if the size of the latter is sufficiently small. With the size of cement grains $4(10^{-7})$ inches in diameter, suggested by the author, a simple calculation, involving the value of surface tension 75 dynes per centimeter, would show that vapour bubbles may extend fully across the void, while tension of 14,000 #/sq. in. or even higher will be maintained in the surrounding water. The voids under these conditions may be all filled with water, with the exception of the spaces occupied by the vapour bubbles. Further desiccation may raise tension in the water even higher, but the vapour space will gradually increase. If this picture of the stresses in the water on drying is correct, there must be a great increase in tension strength of cement associated with drying. The experience, however, disproves this. The writer has done many tension tests of cement involving standard briquets with one square inch in the cross-section. The cement was manufactured by the British Columbia Cement Company. The proportion of mixture by weight was 0.003 : 0.24 : 1 of pozzolith, water and cement respectively. After curing for one month in a moist closet some of the briquets were tested moist, while others were then dried for a week or two in the oven at 150°F. with special precautions to prevent cracking. The strength of the moist briquets was found to be about 700# and of the dry about 1,000# with scattering of the results in both cases not more than 20%. Briquets partially dry showed intermediate values of strength. There was no indication in these tests of the strength approaching even remotely the value of 14,000#.

When tension appears in the water from self-desiccation, a compression of many times, perhaps 100 times, this amount is induced in the grains of cement at their points of contact. The intergranular compression developing in the first few hours after mixing of cement and water undoubtedly accounts for the shrinkage of cement paste on setting, but after this initial stage the paste acquires some hardness and, according to experimental evidence, further decrease in over-all dimensions of the specimen nearly ceases. Yet self-desiccation continues and, as according to the author, the water tension and the intergranular compression are building up, the cement should deform, as it does when it is subjected to ordinary compression by external forces.

The writer has found from experimentation with cement cylinders made of the same proportion of cement paste as the above mentioned briquets, that when tested in one-directional compression a hardened cement cylinder

2. "Soil Mechanics, Foundations and Earth Structures" by G. Tschebotarioff, page 63, McGraw-Hill, 1951.

undergoes at least 0.3% volumetric decrease before failure. At early ages most of this deformation, and later a large part of it, are due to creep or time effect. Why then does not cement keep on shrinking under the action of intergranular compression as it hardens and self-desiccates more and more? Since this compression is triaxial, there is no danger of failure, but the volume decrease may be expected many times 0.3%, if the extremely high value of intergranular compression, inferred from the magnitude of the water tension 14,000#/sq. inch, exists and the creep is in operation. However, according to experience, there is apparently no appreciable shrinkage of cement paste after initial setting, as it is being cured in the moist room.

Deformability of water has a very important bearing on the matters under consideration here. The coefficient of volumetric compressibility of water $\frac{p}{dV/V}$ = 300,000#/sq. inch. If the same coefficient governs also the volumetric dilatation, then the tension stress of 14,000#/sq. inch should result in an increase of the absolute volume of water 4.8%, which corresponds to the increase in the average distances between the molecules of 1.6%. But would the same coefficient be applicable to tension in the water as to compression? It seems clear that the tension coefficient cannot be greater than the compression coefficient, but may very well be smaller, because of the relatively large increase of the distances between the molecules, in which case the volumetric increase of the water under the tension of 14,000#/sq. in. should be greater, even much greater, than the 4.8%.

The magnitude of volumetric dilatation of mild steel in ordinary tension may be mentioned for comparison. At yield point the dilatation is only about 0.05%. Beyond the yield point to failure the volume remains nearly constant, which means that the total dilatation does not increase much above 0.05%. Comparison of water and steel with regard to their dilatations before failure may be considered inappropriate because one body is a liquid while the other is a solid. However, transition of substances, including both steel and water from liquid to solid state, is not accompanied by a large volumetric change, in other words the density of packing of molecules in both states is comparable.

Thus, the large volumetric change occurring in water under tension makes the possibility of existence of a tension stress in it as high as 14,000#/sq. in. highly improbable. Again, looking at the same phenomenon from a different angle, the volumetric dilatation in water will absorb much of the decrease in the volume of water caused by self-desiccation and drying, which will delay the building up of the negative pressure in the water irrespective of how high it may go.

It appears from this discussion that either the tension strength of water is comparatively small or there is something in the conditions present which prevents the high capillary tension in the water from materializing. It is possible that the odd shape of cement grains may prevent formation of regular menisci of necessary curvature, as if the menisci bubbles were getting pricked by the sharp corners of the grains, thus dividing the water into disconnected minute regions, receding on drying to the points of grain contacts.

Even if the water occupying such minute rings is actually at high tension, such tension cannot be measured by piezometer and, being in isolated small regions, it cannot have any effect on capillary flow, in fact it cannot be considered on the same basis as the water either under compression or tension saturating the void spaces. It is incorrect, for example, to speak of the

difference of positive water pressure at one place and negative water pressure of this kind at another as causing rise of water in some parts of concrete dams.

With the water regions becoming sufficiently minute the water ceases to be a liquid, since directional attraction caused by close proximity of solid grains deprives it of its property of equality of normal stresses on different planes at the same point.

The difference in tension strength of dry and moist briquets amounting to some 300# may be attributed to capillary tension at the grain contacts, since some chemically uncombined water remains in cement even after drying at 150°F. However, it is possible also that the increase of tension strength on drying is due to precipitation of sodium, potassium and calcium salts dissolved originally in the water. Although the writer estimates the volume of these precipitated salts as not more than 1/2% of the volume of hydrated cement, if placed in strategic locations this substance may increase the strength of cement materially.

The considerations presented in this discussion lead the writer to the conclusion that the existence of a very high capillary tension in the water present in cement voids, with resulting consequences, is merely a speculation, unsupported and in some aspects strongly contradicted, by the experimental evidence. This statement is not intended to detract from the value of Mr. Powers' excellent paper, presenting in a clear and concise form a scientific opinion, with which the writer does not agree. Simultaneous presentation of the two opposing views will contribute to better understanding of the subject.

J. LAGINHA SERAFIM,[1] A.M. ASCE.—The writer is extremely indebted to Mr. T. C. Powers for his detailed reference to the former's recent experimental findings on the problem of uplift.[2] However the "doubt" as to an "explanation" and a "possible exception" of the results of the tests, stated by the author, deserve comment. As, in a recent discussion of another paper on the same subject,[3] the tests and the apparatus used were described, they do not need to be referred to here.

Firstly, it needs to be stressed that the values of the area factor given in Table I are the mean values obtained in the various tests for three samples of the same mix. For instance, by making the correction for the compressibility of the solid material in the mortar, the three samples of the mix corresponding to Ref. No. 11 gave the following values of the area factor in water tests: (table on following page).

From this the writer concludes that there is not any reason to make exceptions to the results obtained in water tests on the series of samples of compact mortan and on pure cement paste (Ref. No. 10, 11 and 12).

Secondly, when the writer obtained low values of the uplift area in the first water tests, which were carried out after a series of nitrogen tests, he was inclined to admit either that some water inside the pores of the mortar

1. Research Engr., Head of Dam Studies Section, Lab. Nac. Eng. Civil, Lisbon.
2. J. Laginha Serafim - "A subpressão nas barragens" Publ. No. 55, Lab. Nac. Eng. Civil, Lisbon 1954.
3. J. Laginha Serafim - Discussion of Permeability, Pore Pressure and Uplift in Gravity Dams - Proc. ASCE, Sep. 904, Feb., 1956.

TABLE

| Water press. ($kgcm^{-2}$) | SAMPLE 1 | | | SAMPLE 2 | | | SAMPLE 3 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Water test Nº. | Time (hours) | Area factor | Water test Nº. | Time (hours) | Area factor | Water test Nº. | Time (hours) | Area factor |
| 20,2 | 21 | 1/2 | 0,39 | 23 | 1/2 | 0,47 | 25 | 1/2 | 0,44 |
| | | 1 | 0,40 | | 1 | 0,48 | | 1 | 0,43 |
| | | 3 | 0,49 | | 3 | 0,50 | | 3 | 0,33 |
| | | 6 | 0,48 | | 6 | 0,51 | | 6 | 0,36 |
| | | 24 | 0,38 | | 18 | 0,56 | | 24 | 0,42 |
| | | | | | | | | 47 | 0,44 |
| 40,8 | 22 | 1/2 | 0,40 | 24 | 1/2 | 0,43 | 26 | 1/2 | 0,48 |
| | | 1 | 0,39 | | 1 | 0,41 | | 1 | 0,49 |
| | | 3 | 0,40 | | 3 | 0,38 | | 3 | 0,50 |
| | | 6 | 0,43 | | 6 | 0,44 | | 6 | 0,50 |
| | | | | | 24 | 0,43 | | 24 | 0,42 |
| | | | | | | | | 47 | 0,42 |

(adsorbed water films) had an abnormal viscosity or that the procedure used for drying the samples for nitrogen test had caused extensive internal cracking of the cement bonds.

To investigate the first hypothesis the necessity of maintaining the water pressure round the samples for a longer time was evident. The first tests made lasted two hours and then 24 and 48 hour tests were made. As the results obtained were the same, a 19 day test was carried out and this also gave identical results.[4] Such results undoubtedly show that some of the water inside the sample was not able to transmit hydraulic pressures to the solid matter during the time of the tests.

In order to check the second hypothesis, another set of hydrogen tests were made on samples dried in dry air, i.e., without heating them. These nitrogen tests supplied values close to unity and consequently did not differ from these obtained previously. A slight increase in the values of the area factor were however obtained when these same samples were tested again after being dried in an oven at 105°C.

It was noted too that, in the nitrogen tests, the area factor increased slightly with an increase of fluid pressure.

From all these results it was concluded that the water films around the bonds in the cement paste probably do not have mobility. It also appeared probable, in view of the results, that the drying of the mortar, even without heating, causes cracking in the cement paste, thus decisively increasing the value of the area factor in rich mortars. This being the case it seems advisable to continue to adopt an area factor of unity for design.

As a result of the test referred to it was decided to carry out similar tests, with water and nitrogen, on ten cylinders of compact granite. The results of these tests which are practically concluded are very curious. The samples were dried in vacuum and tested with nitrogen and then dried in an oven at 105°C and tested again with nitrogen. After that they were saturated

4. Ref. 2, fig. 7.19.

and tested with water. Such test gave equal values of area factor in nitrogen tests for the two conditions of drying and also equal to those obtained in water tests. The mean values of the results obtained so far are the following:

| Nitrogen tests | Mean value of area factor |
|---|---|
| Cylinders vacuum dried at 20° C | 0.65 |
| Cylinders oven dried at 105° C | 0.68 |
| **Water Tests** | |
| Cylinders saturated by means of vacuum and water pressure | 0.65 |

SERGE LELIAVSKY,[1] M. ASCE.—In the opening paragraphs of his interesting paper Mr. Powers explains that the reason for its production is the "avoidance of conclusions" in the Report of the Subcommittee on Uplift in Masonry Dams, published in 1951.[2] He then proceeds to outline what he believes to be an adequate conclusion.

This means that, in the first instance, since we were prevented (as an exceptional measure) to discuss and criticise this report, such discussion and criticism are now coming to light in the form of, new, apparently independent papers; and in the second instance, it is also clear that since the report in question did not contain constructive conclusions, it was really not a report at all, but merely, a formal list of earlier works on the subject, published by other writers.

Shallow, though, as the report may be it has nevertheless its value (negative?) for it will remain for ever as evidence to the fact that the Uplift problem cannot be solved from elementary considerations, but must be approached on a higher level of engineering mathematics; in fact, it is possibly a significant point that in enumerating the various contributions to the problem, the report of the Subcommittee omits to quote and explain some outstanding instances among its most advanced mathematical solutions.[3]

This last point is materially important in assessing the true value of the author's conclusions. It will be remembered in this connection, that in developing his differential equations, Professor K. V. Terzaghi envisages[3] an infinitesimal element delimited on each of its four sides (in the two-dimensional problem) by surfaces of rupture, which are physically possible in a porous medium, and are, therefore, structurally different from planes or curves capable of elementary symbolical transcriptions. Such a surface of rupture becomes, thus, the cardinal point of the structural basis of the entire theory, and the latter cannot, consequently, be founded on the information on the unbroken micro-structure of the concrete—except in the light of the information on its micro-failure.

It is certainly true that the author of the paper is a well-known authority on the anatomy of the hardened cement paste, but what we, engineers and designers, are interested in, is the pathology of this same paste. This general statement will next be explained in more explicit and specific terms.

1. Civ. and Hydr. Engr., Cairo, Egypt.
2. Trans. ASCE., Vol. 117, p. 1218.
3. See writer's contribution to the discussion of Mr. Carlson's paper on "Permeability, pore pressure and uplift in Dams," in Proceedings Paper 904.

In the first instance the writer wishes to place on record his high appreciation of Mr. Powers' concise, but clear presentation of the splendid work done in the Portland Cement Association laboratories; but, nevertheless, the author cannot agree with Mr. Powers' contention (see 2d. paragraph, page 1) to the effect that the quoted laboratory-work has a direct bearing on the uplift problem in dams. Because, essentially, no uplift tests to destruction were included in the programs of these laboratories. In fact, the author's implicit assumption that the same effective area controls both the deformation of a specimen, on the one hand, and the failure of the dam on the other, is based on nothing.

In the daily routine work of actual (as distinct from imaginary) design work, we are faced with numerous instances proving that the contrary is the case; e.g., the area on which depends the deformation of a reinforced concrete beam is larger than that considered in the calculation of its resistance; the second moment in the buckling-reduction-formula is the gross-moment, whereas the stress in the same member is calculated (at least, according to modern practice) from its net area, etc., etc. Being as they are, fully confirmed by extensive use, these commonly adopted design rules are based on the fact that deflections depend on average areas, whereas failure occurs in the weakest section. This means that in order to be safe and capable, therefore, of being used in design, the uplift factor must be derived from tests on destruction, which depend on the weakest section; and not on tests on deformation of the specimen which must of necessity yield the average area. In that sense, consequently, the author's assumption is not satisfactory.

Thus, in so far as micro-structure is concerned, the true, design-worthy percentage of effective uplift area must be more in the way of an exception, rather than an average. This conclusion cannot be over-estimated, for recent information (both in America and in Egypt) tends to show that the Gaussian frequency curve, which was heretofore believed to be in generally good agreement with the facts derived from Nature, is in striking disagreement with such facts, in the two extreme zones of the range (see hatched areas in Fig. 1).

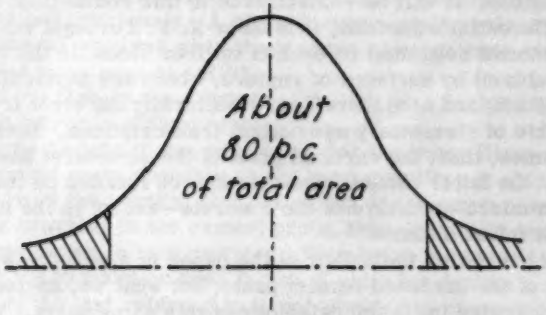


Fig. 1

It is therefore extremely probable that average conditions—such for instance as the imaginary twelve points of contact of a gel particle—so aptly described by the author, have no bearing whatever on the exceptional

conditions controlling an uplift failure. This point will be even more evident, if the author's conventional assumptions are viewed in the light of the information supplied by micro-photography, which shows that the structure of concrete resembles the complicated pattern of an archipelago, rather than the classical simplicity of a Greek temple.[4]

In view of these facts it is possibly surprising that extensive experience with destruction uplift tests—such for instance as those carried out by the writer[5]—tends nevertheless towards a consistent value of the uplift factor and this in spite of all steps taken to provoke differences in micro-structure. It is fairly obvious that such consistent results could never have been obtained, had this factor been dependent on the vagaries of the concrete micro-structure.

An explanation of this surprising consistency has first been suggested by Professor Raes[6] and later been developed and popularised by the writer,[3] viz. that any uplift factor is possible in any micro-structure, depending on the geometry of the micro-failure. The fact, therefore, that destruction tests yield consistent results, is due to a general physical law which controls micro-failure (as distinct from micro-structure). How this is possible is explained in Figs. 2a and 2b.

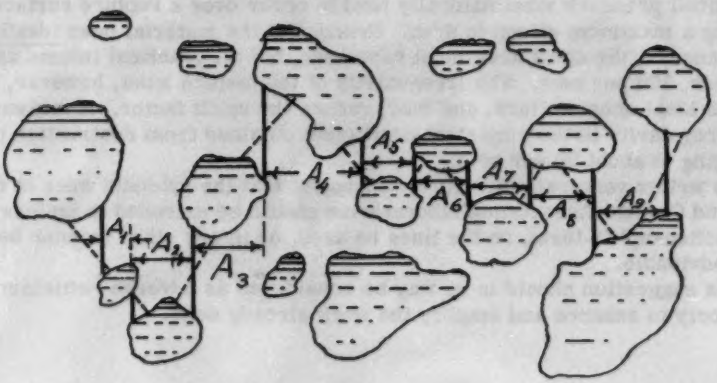


Fig. 2a

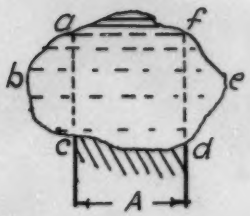


Fig. 2b

4. See, for instance, the electron-microscope-photograph by Professor N. Hast, reproduced on page 2 of the Report 57 presented to the IIId Congress on Large Dams, Stockholm, 1948, by Mr. Erling Reinius.
5. Serge Leliavsky Bey, "Experiments on Effective Uplift Area in Gravity Dams, "Trans. Amer. Soc. C.E., Vol. 112, 1947, page 444 et seq.
6. Trans. Amer. Soc. C.E., Vol. 114, 1949, p. 266 and foll.

The micro-structure in Fig. 2a is an intentionally choosen irregular pattern, and what is even more important, the conclusions derived therefrom do not depend on this particular pattern (as would have been for instance the case, had the author's, classically ordained, twelve-point pattern been used as a basis for discussion).

It will be clear from the figure that a pure-shear failure which is here indicated by vertical lines, yields a 100 per cent uplift factor. Should any doubt exist on this point, a reference to Fig. 2b will suffice to confirm the validity of the principle; in fact, the pressures on ab and cf being balanced by the pressures on bc and de, the net resultant is the pressure on af i.e. on A; and as seen from Fig. 2a the sum $\Sigma$ (A) is always 100 per cent.

Q.E.D.

On the other hand, remembering that in this case the solid material is an ideal solid, it follows that the inclined lines in the same drawing, which represent a shear-cum-tension failure, correspond to a lesser percentage of uplift area, depending on the slope of these failure lines.

The slope of the surfaces of failure under mechanical load is frequently determined by Mohr's critical lines, or similar criteria, but failure due to interstitial pressure must naturally tend to occur over a rupture surface yielding a maximum effective area. Hence, had the material been ideally homogeneous, the uplift area must have been, for all practical intents and purposes, 100 per cent. The irregularity of the texture must, however, affect the ideal shear failure, and thus, reduce the uplift factor. A measure of this irregularity is the empirical coefficient obtained from destruction tests, averaging to about 90 per cent.

The writer ventures, therefore, to suggest that the splendid work of the Portland Cement Association laboratories should be extended to include destruction-uplift-tests, on the lines he used, or in any other manner believed to be advisable.

This suggestion should in no way be considered as adverse criticism but tends only to enhance and amplify the work already done.

## PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

### VOLUME 81 (1955)

FEBRUARY: 608(WW), 609(WW), 610(WW), 611(WW), 612(WW), 613(WW), 614(WW), 615(WW), 616(WW), 617(IR), 618(IR), 619(IR), 620(IR), 621(IR)[c], 622(IR), 623(IR), 624(HY)[c], 625(HY), 626(HY), 627(HY), 628(HY), 629(HY), 630(HY), 631(HY), 632(CO), 633(CO).

MARCH: 634(PO), 635(PO), 636(PO), 637(PO), 638(PO), 639(PO), 640(PO), 641(PO)[c], 642(SA), 643(SA), 644(SA), 645(SA), 646(SA), 647(SA)[c], 648(ST), 649(ST), 650(ST), 651(ST), 652(ST), 653(ST), 654(ST)[c], 655(SA), 656(SM)[c], 657(SM)[c], 658(SM)[c].

APRIL: 659(ST), 660(ST), 661(ST)[c], 662(ST), 663(ST), 664(ST)[c], 665(HY)[c], 666(HY), 667(HY), 668(HY), 669(HY), 670(EM), 671(EM), 672(EM), 673(EM), 674(EM), 675(EM), 676(EM), 677(EM), 678(HY).

MAY: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(ST), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

JUNE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

OCTOBER: 809(ST), 810(HW)[c], 811(ST), 812(ST)[c], 813(ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

### VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1) 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877 (HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 (PO1)[c], 905(SA1)[c].

c. Discussion of several papers, grouped by Divisions.

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